CONCRETE AND CONSTRUCTIONAL ENGINEERING

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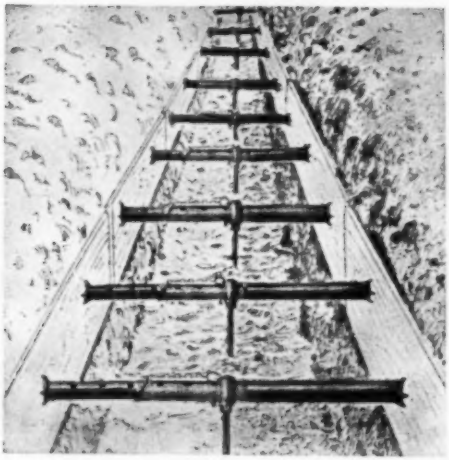

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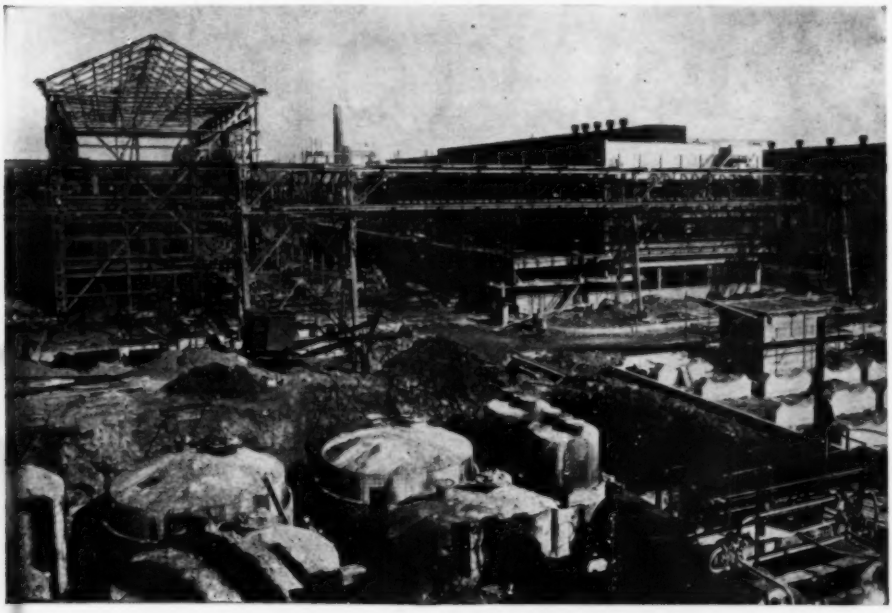

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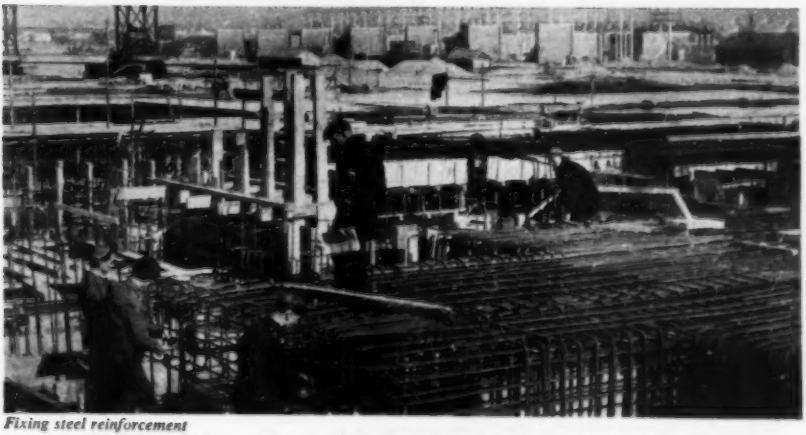

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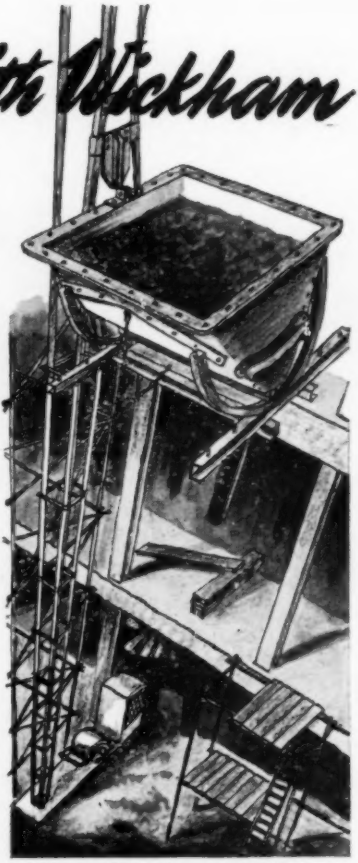

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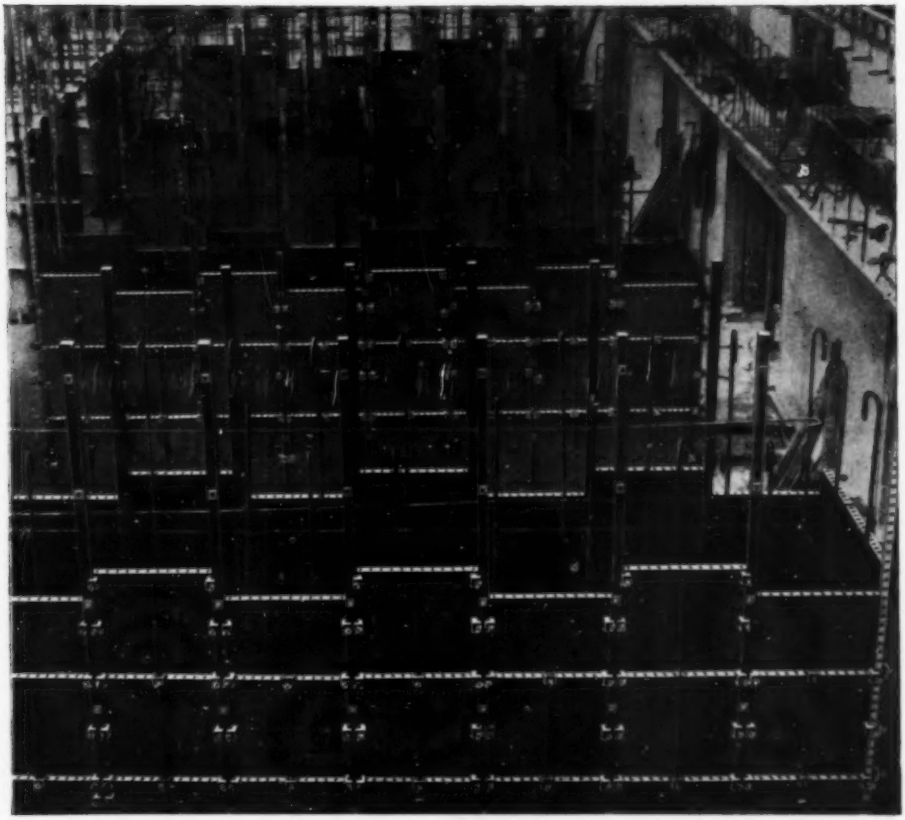

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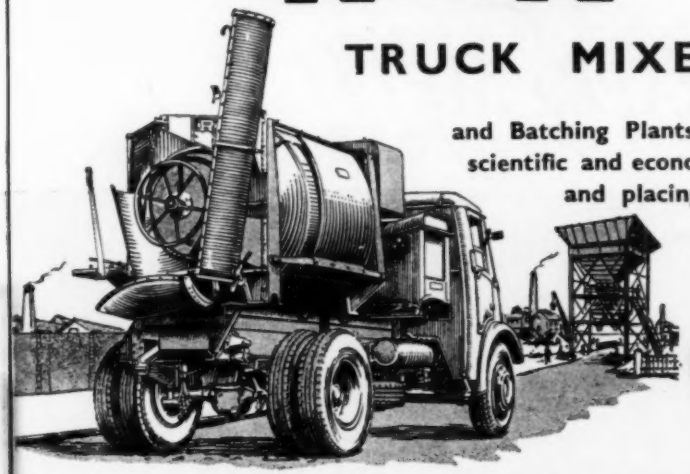

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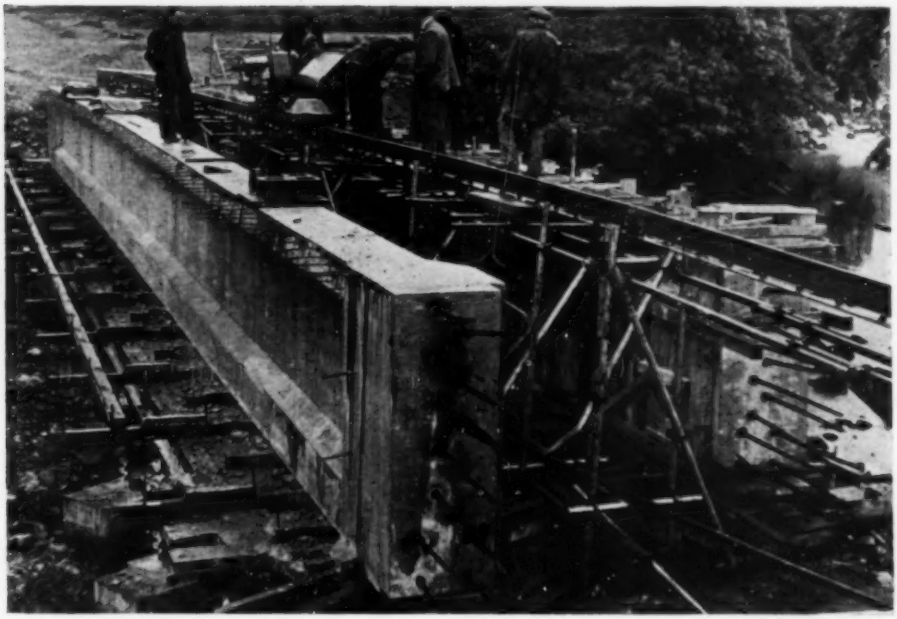

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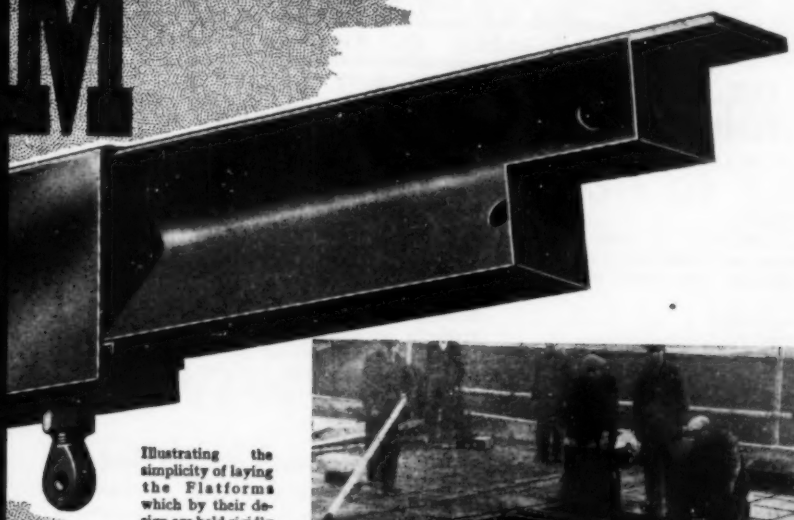

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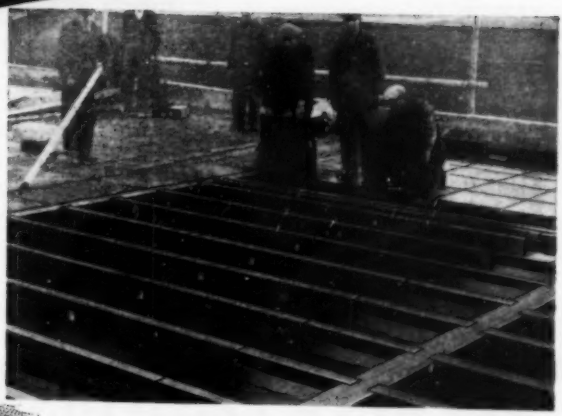

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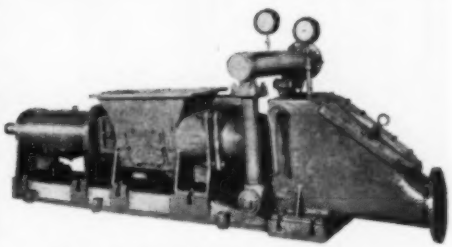

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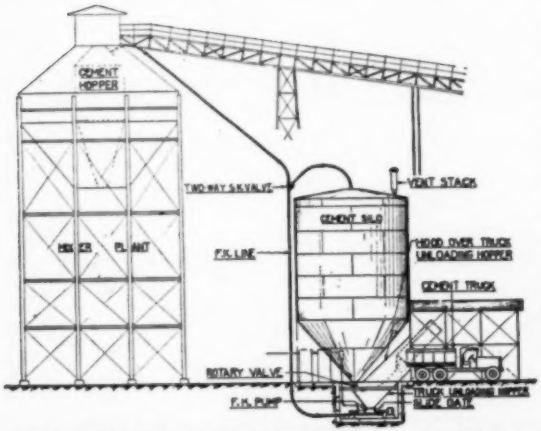

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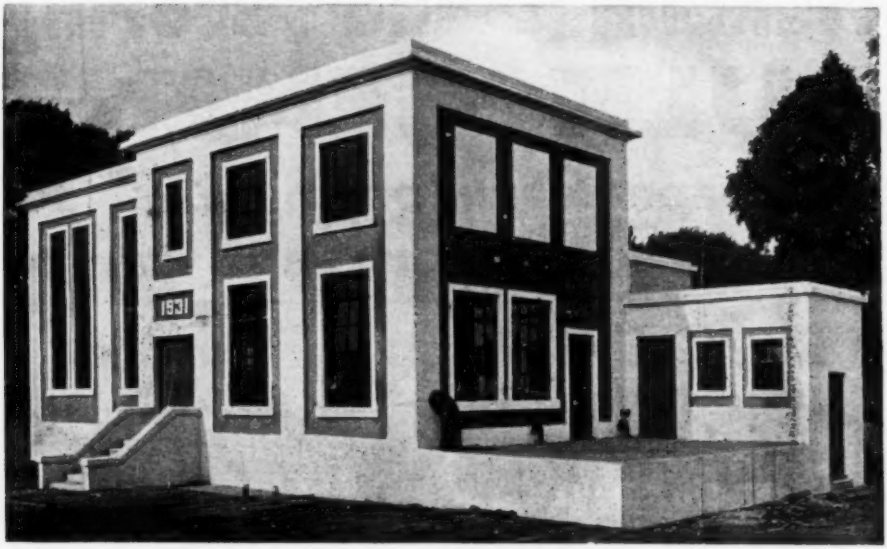

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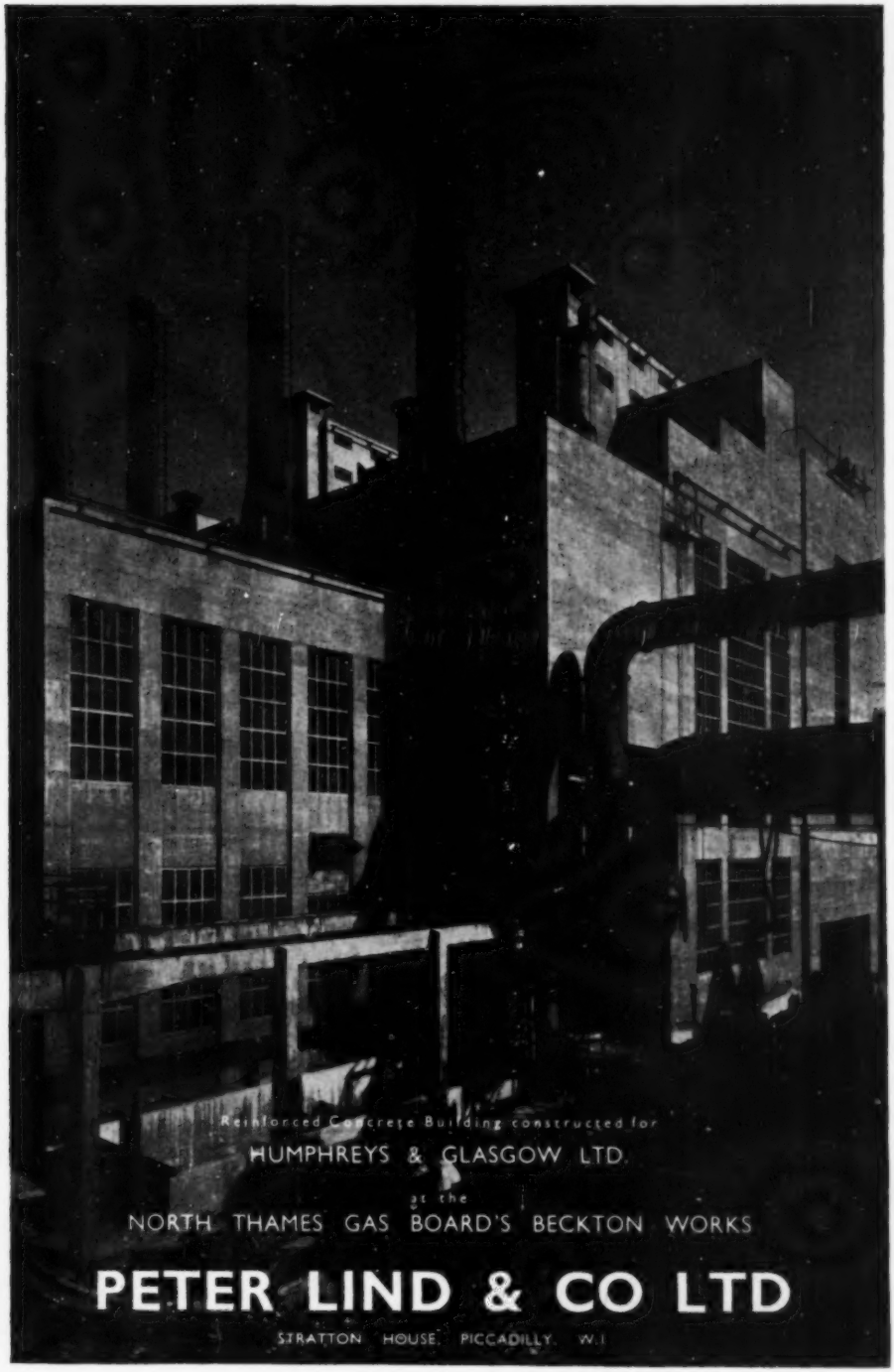
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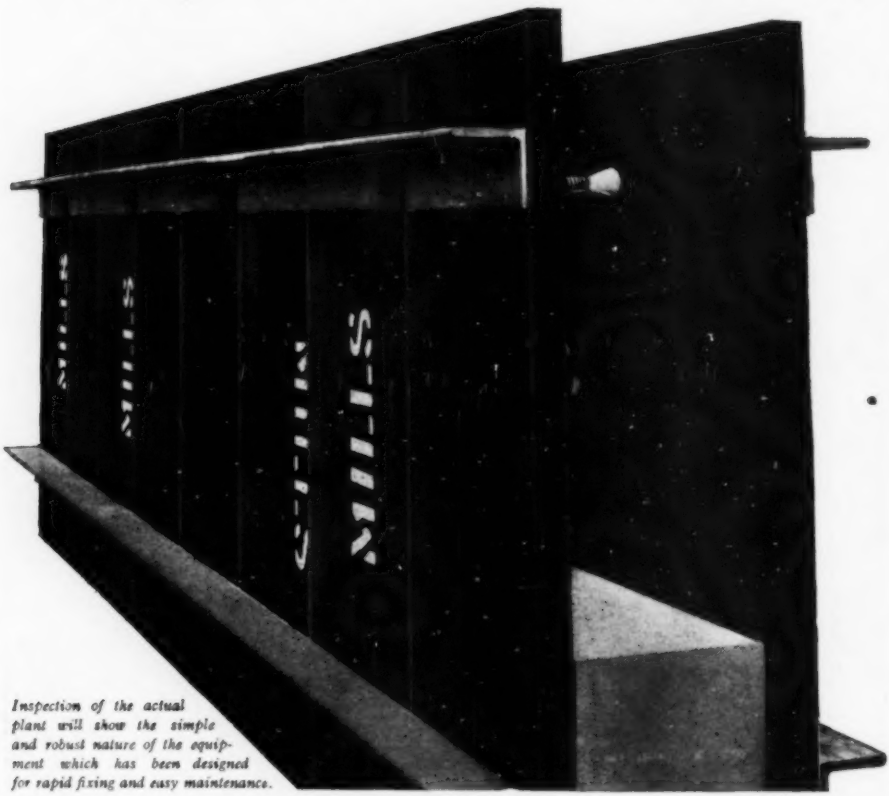

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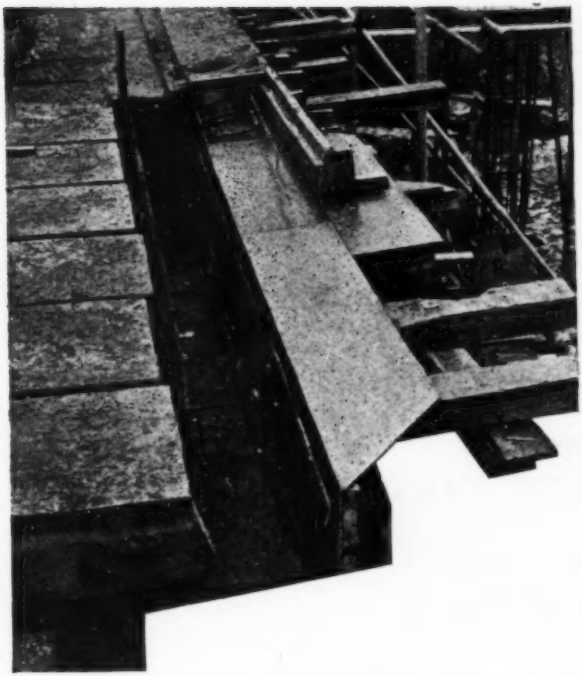

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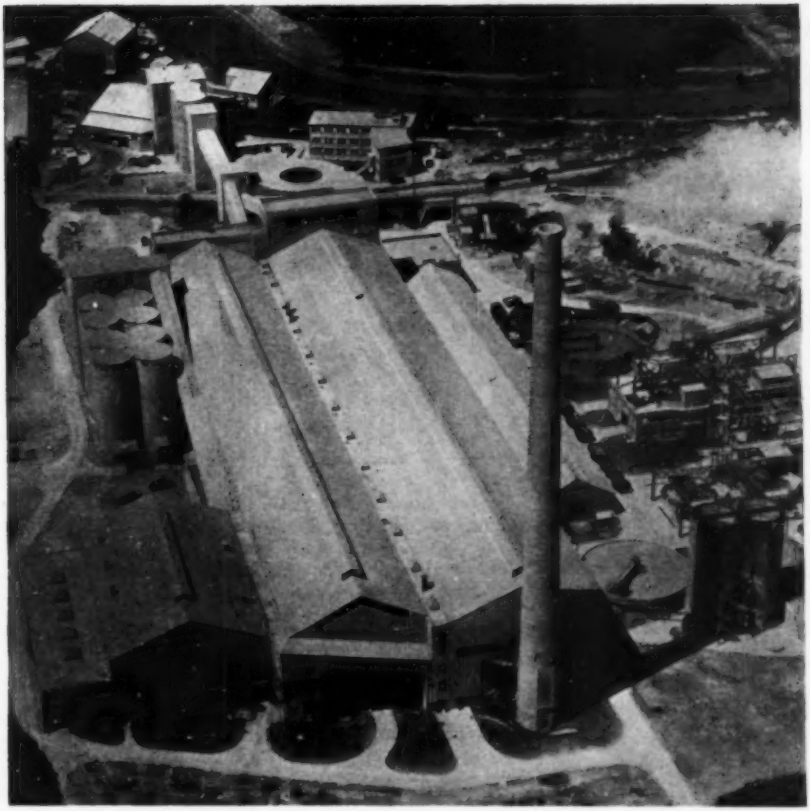

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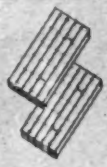

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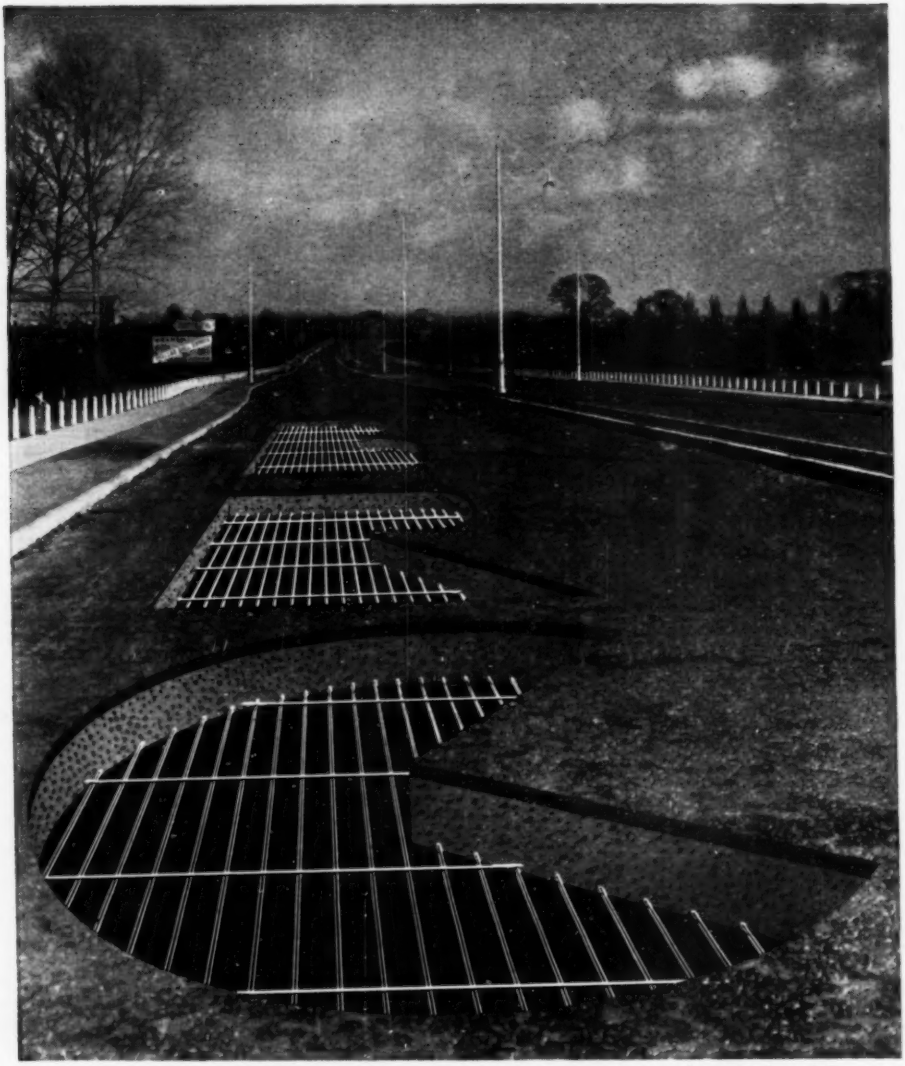

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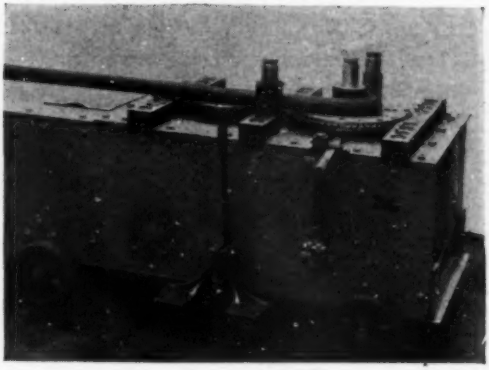

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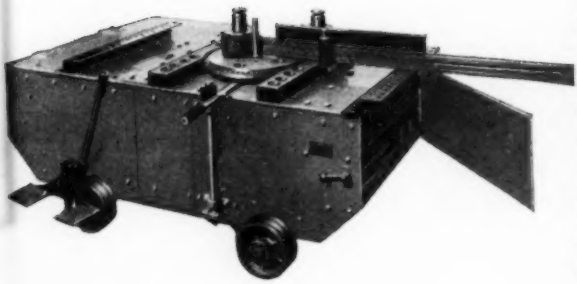

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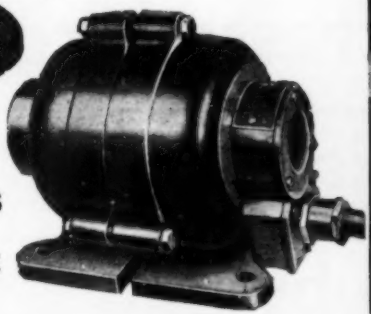

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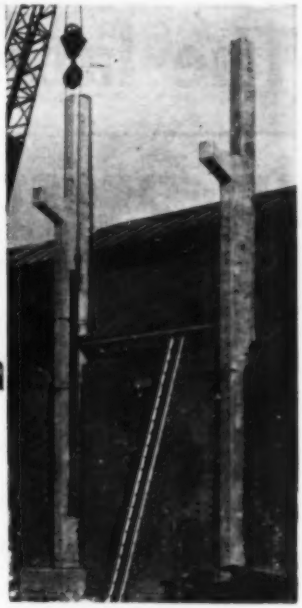

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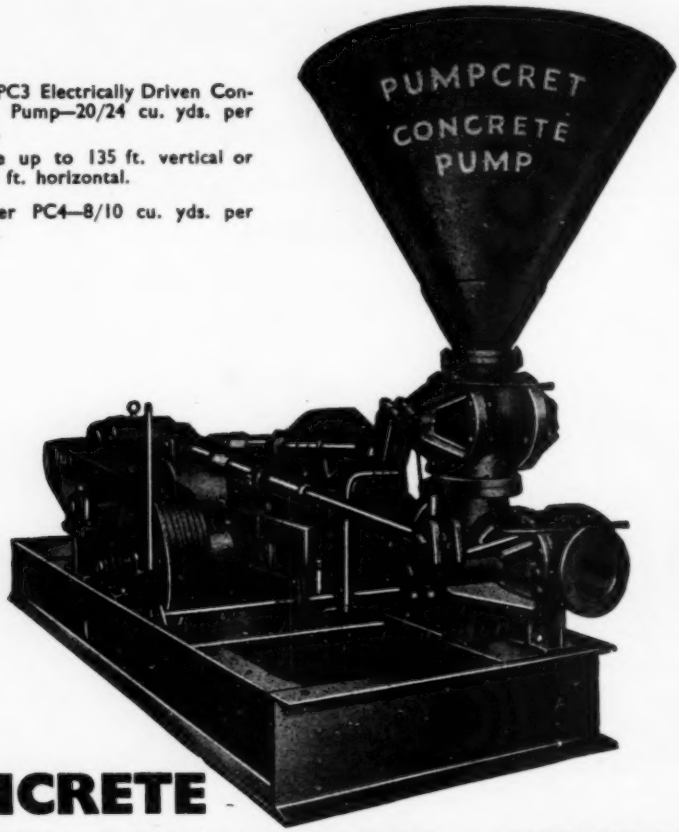

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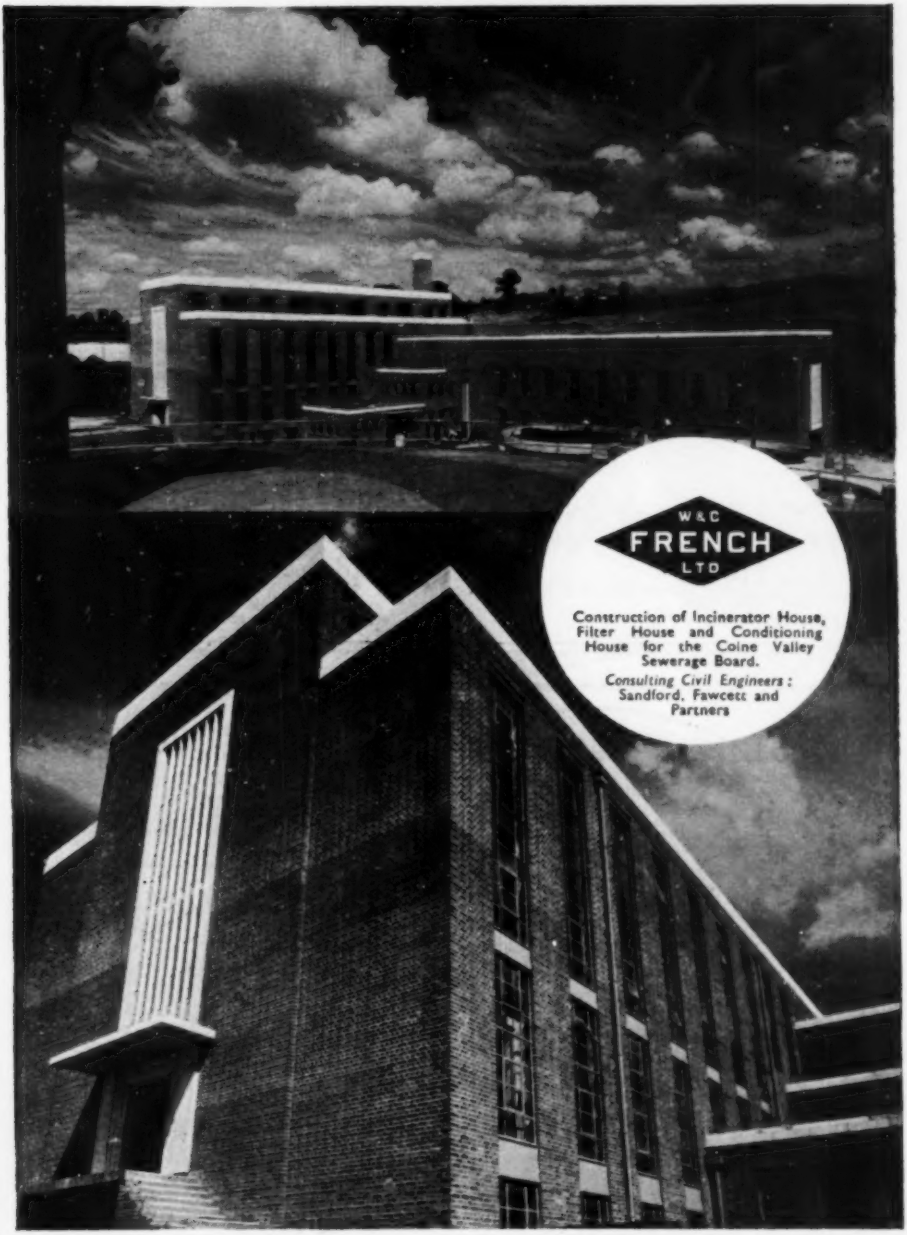
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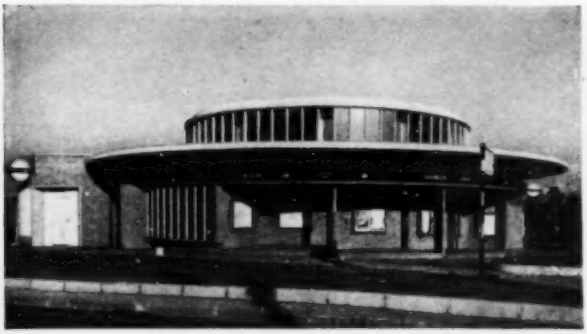

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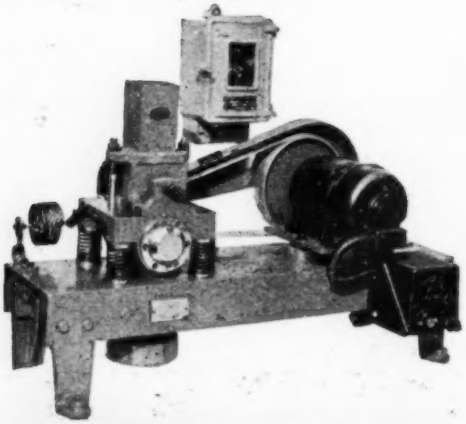

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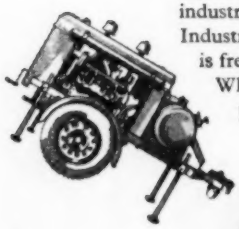

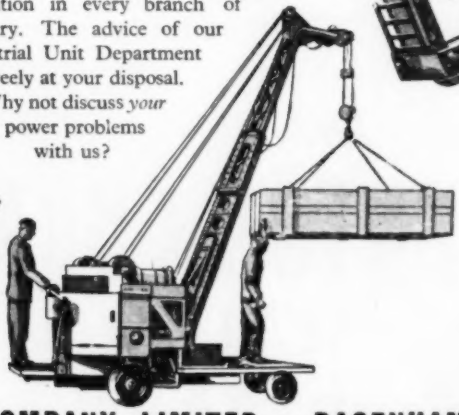

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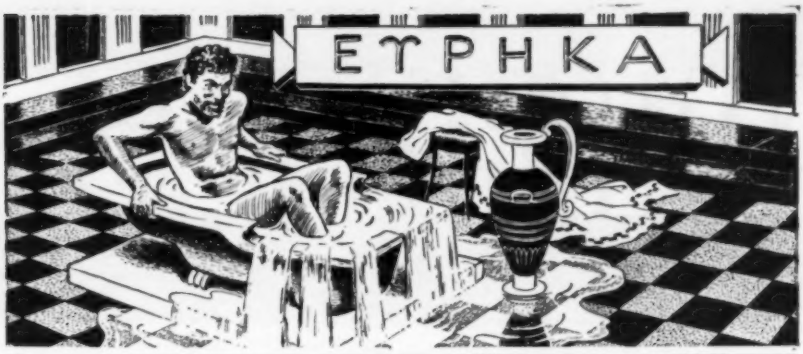

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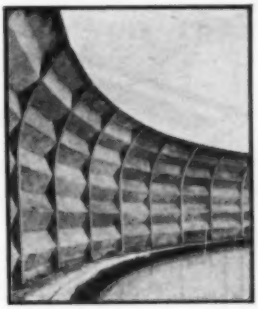

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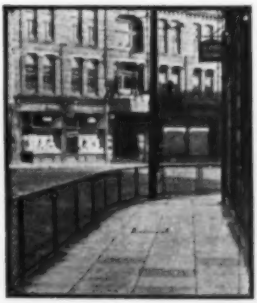

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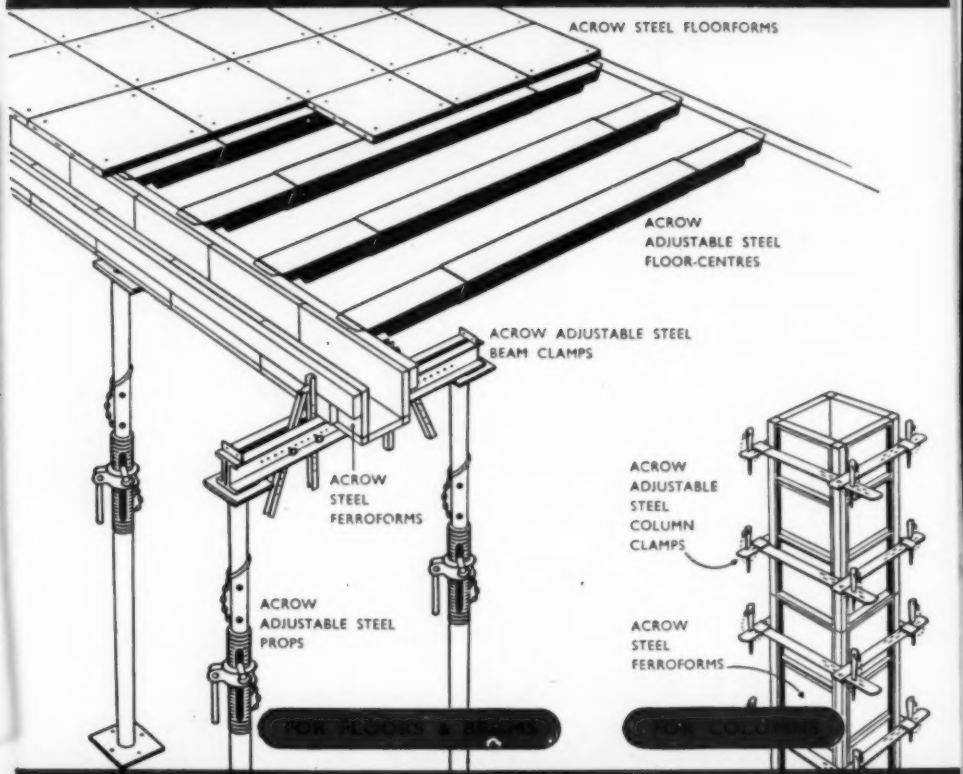

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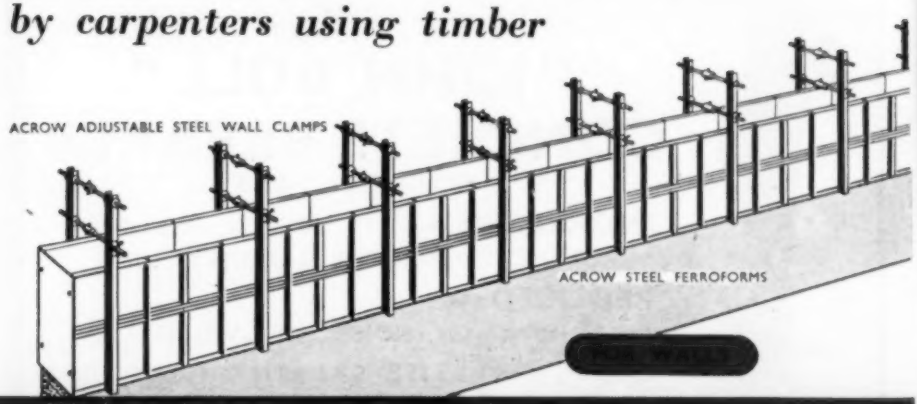

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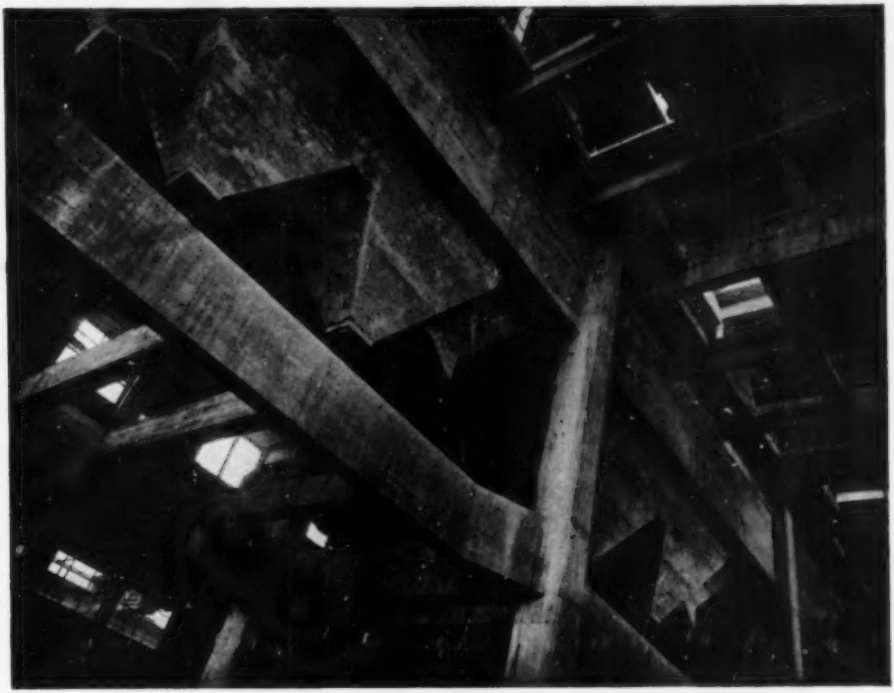

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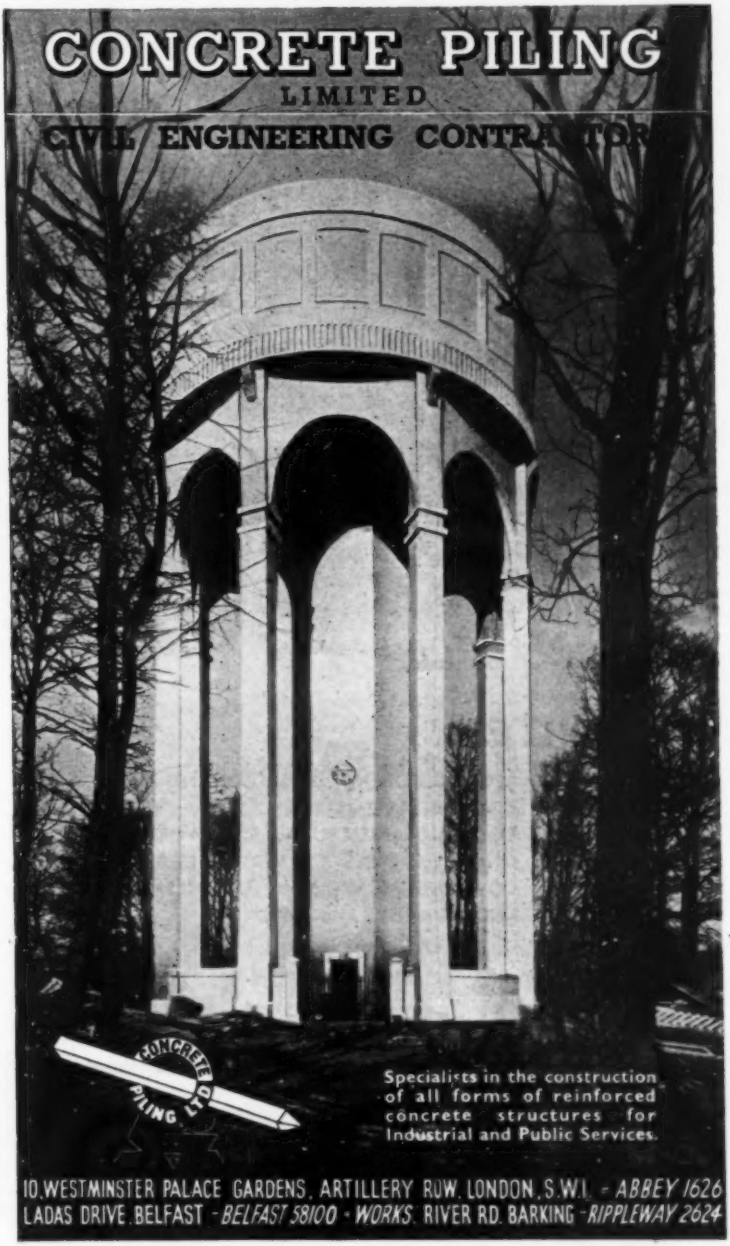
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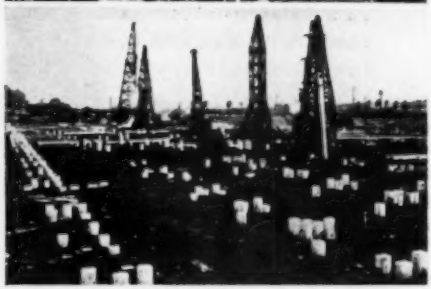

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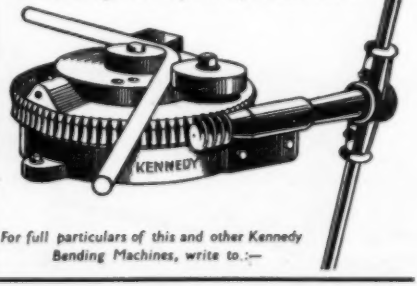

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EDITORIAL NOTES

The Theory of Plastic Hinges.

A THEORY of the behaviour of structures, which has been developed during the past decade, is that the failure-load of a statically-indeterminate structure can be determined by the introduction of " plastic hinges ". Briefly, the theory is that, as the load on a structure increases, the stress at one section of a member will become equal to the yield stress of the material and the elastic condition is superseded by a plastic condition. Consequently resistance to bending ceases to increase at the section, which acts virtually as a partial hinge, the moment of resistance of which is limited to the least bending moment that causes plastic yield. As a result the bending moments on the member concerned are redistributed, and the redistribution may extend throughout the structure. As the load increases further, the plastic condition is reached at another section, the bending moments are again redistributed, and so on until the structure fails. The ratio of the load required to cause failure to the working load is one measure of the safety of a structure, but development of plasticity is accompanied by comparatively large strains ; consequently the deformation due to the formation of the first hinge, or of any subsequent hinge, may be so great that the structure would no longer serve its purpose and the ratio of the load required to cause the critical hinge to the working load is the real criterion of the strength of the structure.

The conception of partial hinges, which makes easy the analysis of highly statically-indeterminate structures, may be attractive to designers of monolithic reinforced concrete frames, but it is beset with practical difficulties that may not be immediately evident. The theory must not be confused with the plastic or ultimate-load theory of the resistance of reinforced concrete beams, nor is the redistribution of the bending moments exactly the same as the redistribution that takes place because of the greater creep of the concrete at sections where, under working load, the compressive stress in the concrete is greatest. To allow for the latter redistribution it is common to provide resistance at midspan of a continuous reinforced concrete beam greater than that required by formulæ based on the elastic theory and to decrease correspondingly the resistance at sections near the supports. It is important to draw a distinction between ordinary elastic methods of design at working loads and ultimate-load methods.

The plastic-hinge theory as generally expounded is primarily intended for steel structures, but how far it is applicable to reinforced concrete is, in the present state of knowledge, problematic. A feature of the plastic-hinge theory that may

appeal to many engineers is the fact that it is based on the properties of the materials and not entirely on the abstract mathematical considerations inherent in the elastic theories. If design is to be based on the properties of a material, however, it is obvious that the properties must be known. The strains occurring in reinforced concrete are not well established for all likely conditions, and postulating the position of the first and subsequent hinges is not easy although it may be so in a metallic structure. Consider in this respect the difficulty of calculating the bending moments that would just cause plasticity in the common case of a reinforced concrete tee-beam (the width of the compressive flange of which is indefinite) and in the same beam with the flange subjected to tension and with compressive reinforcement in the bottom of the rib. Mild steel, either in a steel structure or as reinforcement in a concrete structure, is entirely elastic at the working load and does not generally reach a plastic stage until the stress is about twice the design stress. Concrete in compression may, however, exhibit plastic yield, or permanent deformation, simultaneously with elastic strain (or temporary deformation) at stresses of the same magnitude as the design stresses. The difficulty of determining the moment of resistance of a plastic hinge in reinforced concrete is therefore evident. In a rectangular reinforced concrete section the concrete adjacent to the compressive edge may attain plasticity while that nearer the neutral axis is still mainly in an elastic condition. This condition is not so pronounced in a steel I-beam.

No theory or speculation of the behaviour of a structural member or an assembly of members is likely to be quite correct. First, there are the unpredictable vagaries of the material due, in the case of concrete, to stress, age, creep, moisture and thermal movements, fatigue, variation of workmanship, and so on. Equally important are the numerous conditions that have to be assumed to make a structural analysis capable of being applied to design. The theory which embodies the fewest assumptions may be more exact, but it may be too cumbersome to be usable. If the results obtained by the application of formulæ based on a theory agree with the results of tests, the correctness of the theory is not necessarily proved. If agreement is obtained over a wide range of tests, it can be said that the theory results in a good working rule. Such theories are common in reinforced concrete, and practice has shown that they can be used satisfactorily in design although known to be based on erroneous assumptions. One inherent difficulty is that the stresses in a statically-indeterminate structure before the application of the load are not known. A common assumption, and one that is without doubt far from the truth, is that the unloaded structure is not strained, whereas construction story by story or bay by bay produces unknown initial bending moments and stresses in the new concrete which are not represented in ordinary analyses. Shrinking of the concrete also causes unknown stresses, which should be included in a calculation of the total stresses upon which the production of a state of plasticity depends. Considerable research is therefore necessary before the plastic-hinge theory can be applied with confidence to the design of a reinforced concrete structure.

Substructure and Oil Store at Electricity Works.

THE new power-house at the new electricity generating station at Bankside, London, is a steel-frame structure with brick walls, but there is much concrete construction of interest in the substructure and ancillary works. About half of the power-house is now in course of construction ; it will have oil-fired boilers and will eventually contain four 60,000-kw. generators. The oil will be discharged from tankers at the wharf (described in this journal for August, 1951) and pumped through pipes to a store at the rear of the power-house. The cooling water flows from the river through the intake tunnel to the deep suction chamber behind the chimney-base in the power-house.

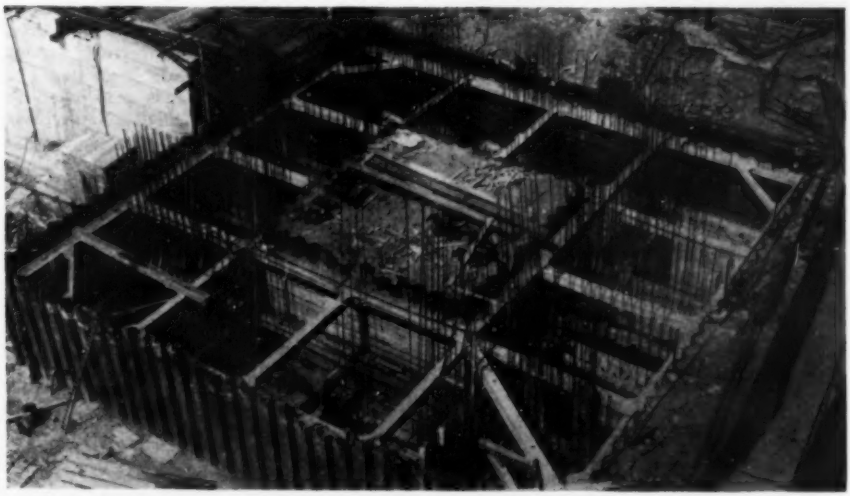


Fig. 1.—Permanent Reinforced Concrete Struts and Walings in Excavation for Chimney Base.

Substructure of the Power-House.

The part of the power-house now being constructed is almost surrounded by a gravity retaining wall of 1 : 6 plain concrete. The length of the wall along the front and back of the building is about 275 ft., and it extends along one end of the boiler house (103 ft.), turbine house (85 ft.), and switch house (56 ft.). A temporary concrete wall and a cut-off wall extend along the fourth side, where it is proposed to extend the power-house later. The thickness of the main retaining wall at the ground floor (+ 18·5 ft.) is 7 ft. and increases to 10 ft. at the basement level (— 9·5 ft.), below which it is 16 ft. thick. The basement floor is 18 in. thick and is generally of plain concrete, but is heavily reinforced in parts, for example at ducts and around the chimney base. The wall and the foundations for stanchions and turbine blocks are carried down through silt and ballast to London blue clay, which occurs at levels varying from — 18 ft. to — 30 ft. and on which the bearing pressure is generally about 3 tons per square foot but does not exceed 4 tons per square foot. The foundations for the west annexe, which has no basement, are 12-in. diameter bored piles concreted under pressure.

The water table is at about — 3 ft. The retaining wall was constructed in a trench within a steel sheet-pile cofferdam. Steel-faced timber shuttering was used.

The piling around the outside of the wall remains in position but that on the inside was extracted after the dumpling had been excavated. For the chimney foundation the depth of the excavation is about 52 ft. below + 18·5 ft. Because of the difficulty of removing ordinary timbering, support for the sides of the excavation was provided by reinforced concrete walings and struts (*Figs.* 1 and 2) most of which were later built permanently into the cellular structure of the chimney foundation, which incorporates the end of the cooling-water intake tunnel and penstock shaft. The concrete was placed by pump, and timber shuttering was generally used.

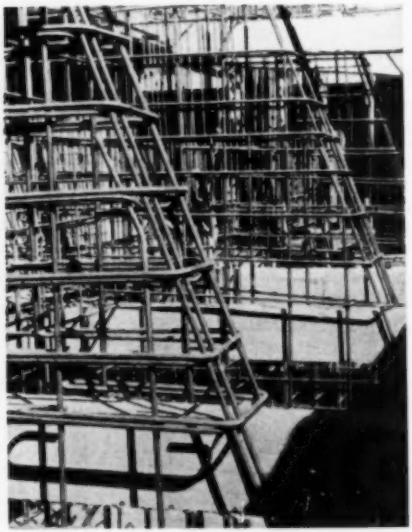


Fig. 2.—Permanent Reinforced Concrete Struts and Counterforts.

The Terrace.

At the rear of the power-house there is, at + 17·75 ft., an elevated reinforced concrete access road, or terrace, about 300 ft. long and 55 ft. wide (*Fig.* 5). The space below the terrace is excavated to − 14·0 ft. and is paved with a 12-in. slab. The terrace is designed for the heaviest Ministry of Transport loading, which is 135 tons from one vehicle. The 10-in. deck slab is in panels 13 ft. 6 in. square. The transverse supports are reinforced concrete beams 24 in. deep spanning about 13 ft. 6 in. and supported on longitudinal steel lattice girders about 8 ft. deep encased in concrete. The steel girders,

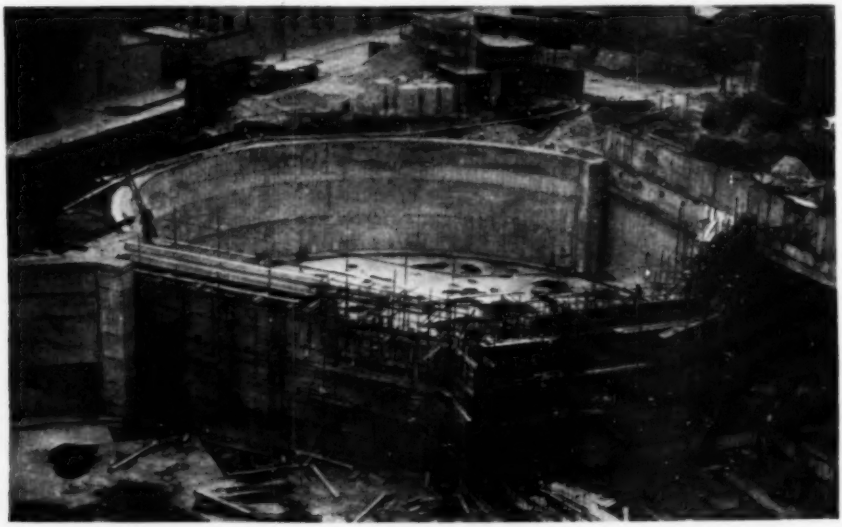


Fig. 3.—Oil Store.

which were provided because they could be supplied without delay and facilitated the erection of the shuttering for the terrace, span 35 ft. to 41 ft. between 12-in. reinforced concrete walls, which have pilasters under the bearings of the girders, and extend from the deck to a foundation block of plain concrete at − 14·0 ft. The foundation is 7 ft. wide and extends at least 6 in. into the clay. One end of the terrace is seen on the right-hand side of *Fig.* 3.

the slab is generally expanded metal, but 1-in. bars at 9-in. centres are provided in two directions under the steel columns supporting the reinforced concrete roof of the tank.

The columns, which are set out on a grid of 17-ft. squares, have flared heads, and the 12-in. roof slab is designed as a flat slab carrying earth (sown with grass) and an imposed load of 100 lb. per square foot. The slab extends beyond the sides of the steel tank to a support on the ex-

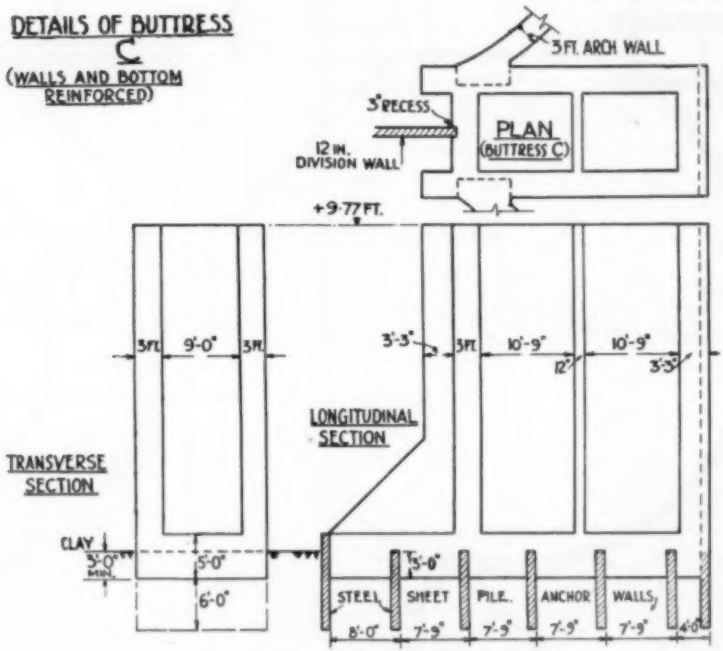


Fig. 4.—Buttresses of Oil Store.

Oil Store.

The oil store is adjacent to the terrace. The oil will be contained in three steel tanks each of 1,000,000 gallons capacity. The tanks, which are 95 ft. diameter and 25 ft. deep, are below ground in a pit of roughly clover-leaf shape in plan (*Figs.* 3 and 5). The bottom of the pit is at − 13·0 ft., and is a 12-in. concrete slab the top of which had to be laid level within $\frac{1}{8}$ in. because the steel plates forming the bottom of the tank are laid directly on the concrete. The reinforcement in

ternal walls and division walls (*Fig.* 5) of the pit. The external walls are retaining walls acting as large horizontal arches spanning between four buttresses.

The buttresses (*Fig.* 4), which are in line with the division walls between the tanks, are reinforced concrete boxes 15 ft. wide, generally 43 ft. long at the bottom and 32 ft. 6 in. at the top at + 9·77 ft. The walls of the buttresses are generally 3 ft. thick and the cells are filled with consolidated material. The bottom is 5 ft. thick and is taken at least 3 ft. into the clay, but, to increase the resistance

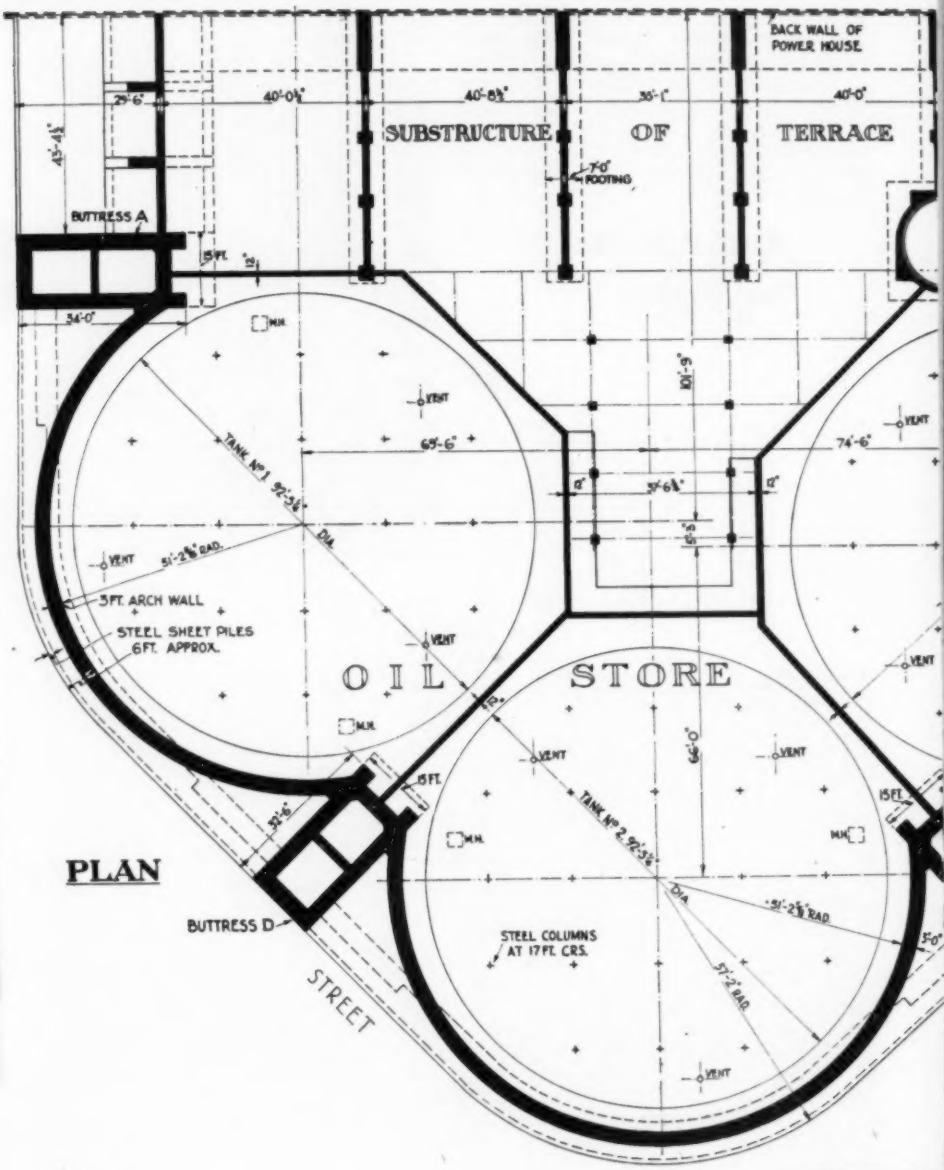
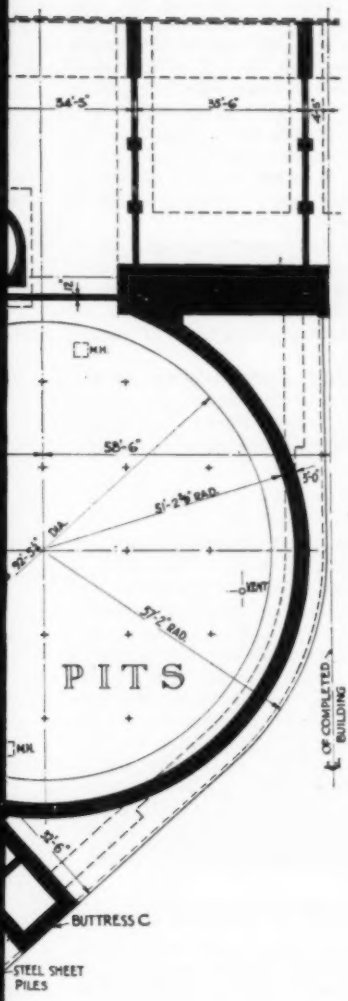
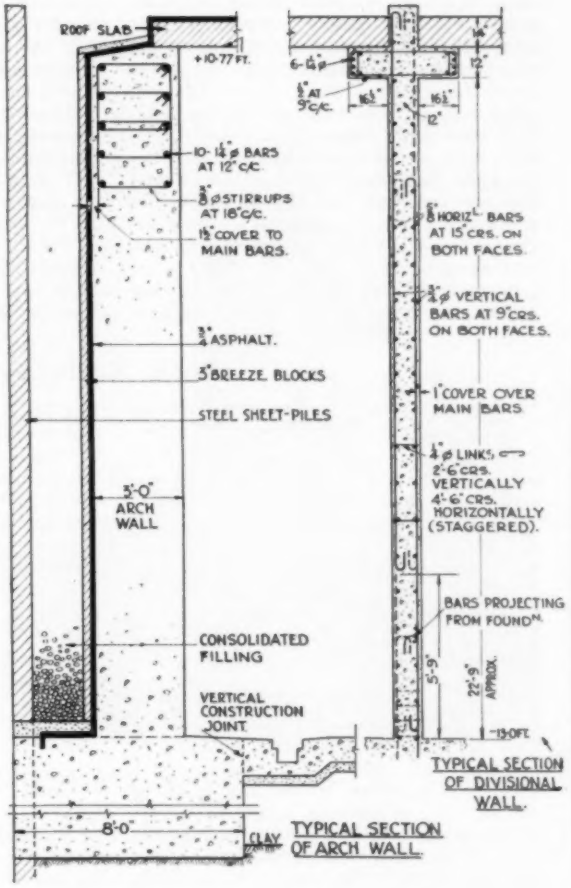

BACK WALL OF POWER HOUSE
SUBSTRUCTURE OF TERRACE
7'-0" FOOTING
BUTTRESS A
15 FT.
34'-0"
29'-6"
40'-0½"
40'-8½"
35'-1"
40'-0"
43'-4½"
M.H.
VENT
TANK No 1 92'-3½" DIA.
51'-2½" RAD.
3 FT. ARCH WALL
STEEL SHEET PILES 6 FT. APPROX.
65'-6"
101'-9"
74'-6"
35'-6½"
12"
6'-9"
OIL STORE
66'-0"
TANK No 2 92'-3½" DIA.
51'-2½" RAD.
57'-2" RAD.
STEEL COLUMNS AT 17 FT. CRS.
BUTTRESS D
STREET
PLAN

Fig. 5.—Oil





e and Terrace.

to horizontal thrusts from the arch walls, seven transverse rows of steel sheet-piles are driven 6 ft. into the clay below the bottom of the buttress and embedded 3 ft. in the bottom slab.

The span of each circular-arch wall is about 110 ft., and the construction is that, below the wall from − 14·0 ft. down to clay, there is a concrete cut-off wall 8 ft. thick (*Fig.* 5). Above − 14·0 ft., the arch wall is of 1 : 2 : 4 concrete 3 ft. thick, and without reinforcement except for a depth of about 4 ft. 6 in. at the top where a ring-beam is formed by five 1¼-in. bars near both faces of the wall. The back face of the cut-off wall extends beyond the back face of the arch wall, and was cast against a line of steel sheet-piles which are left in position. The space between the sheet-piles and the arch wall was filled with consolidated filling after ¾ in. of asphalt protected by 3-in. clinker blocks had been applied to the back of the arch wall. The shuttering for the arch walls was three lifts of narrow rectangular steel plates placed with the long sides vertically. The shuttering for the division walls (*Fig.* 3) was timber panels in some cases, and in others the steel plates used for the arch walls but placed with the long sides horizontally. The concrete was placed by a pump at the mixing plant seen in the background of *Fig.* 3. The cement was supplied in bottom-opening steel cylinders each of which contained 2½ tons.

The roof of the power-house is a 4½-in. concrete slab reinforced with mild steel bars and supported on steel purlins at 8 ft. 6 in. centres. The roof lights are of glass-concrete construction in panels 25 ft. square.

The consulting civil engineers to the British Electricity Authority are Messrs. Mott, Hay and Anderson, and the contractors are Sir Robert McAlpine & Sons, Ltd.

Book Reviews.

"Das Beton A.B.C." By A. Hummel. Eleventh edition. (Berlin. Wilhelm Ernst & Sohn. 1951. Price 16 DM.)

THIS is more than an introduction to concrete. It is a complete textbook for those responsible for the control of concrete, and includes a survey of research, the results of which are summarized in the form of rules relating to materials, mixing, placing, compaction, and testing. Lightweight concrete is dealt with fully. There are 243 pages of text, 17 tables, and 126 illustrations.

"Cálculo, No Regime de Ruptura, Das Lajes de Concreto Armadas em Cruz." By S. Chamecki. (San Paulo: Editora Guaira, Lta. 1948.)

THE tendency in structural analyses to consider the load causing failure of a structural member to be the basis of design, with a suitable factor of safety, is discussed, and the author develops a method of design for reinforced concrete slabs spanning in two directions based on their state at failure under different loadings and conditions of support. Tables enable much arithmetical work to be avoided, and examples show how economical designs can be prepared. Agreement between theory and practice is shown by comparison with the results of numerous tests made in Europe. The magnitude of the factor of safety required under different conditions is considered. The book is in the Portuguese language, and comprises 100 pages. The method by which design is based on the pattern of the cracks in a slab at failure is a revival of methods put forward many years ago, and is being given attention by many writers. The obvious inapplicability to a reinforced concrete slab of the assumption in the elastic-plate theory that the slab is an infinitely thin plate, and the difficult mathematical procedures associated with this theory, make it understandable why a simple theory based on the actual behaviour of a slab should appeal to many engineers.

"Telai Elastici." By C. L. Brunoli. (Milan: Ulrico Hoepli. 1951. 200 pp. Price 2000 lira.)

THIS book deals with a method of analysis of elastic framed structures based on the position of nodes. As in most purely analytical methods the results depend upon the solution of several simultaneous equations, and one method described is that of Banachiewicz which is applied in an example to the solution of eleven equations; if the rules are mastered the procedure is not complex but takes much space. Much of the book deals with Vierendeel girders, and there is a section on torsion on horizontal frames. There are sixteen numerical examples.

A Large Jetty on Southampton Water.

HOLLOW PILES 32 IN. DIAMETER.

THE marine terminal (*Fig.* 1) of the new oil refinery for the Esso Petroleum Co., Ltd., on Southampton Water at Fawley comprises principally an approach nearly 200 ft. long and a pierhead 3200 ft. long (*Fig.* 2). The general arrangement was planned by the Standard Oil Development Corporation, who also designed the approach. The detailed design of the jetty and the adjoining pumphouse area is by Messrs. Christiani & Nielsen, Ltd., who also constructed the approach, the pierhead, and the pumphouse area. The buildings and equipment, including pumps, pipes, and hose-handling structures, were designed and installed by others.

The structures contain 637 rectangular reinforced concrete piles, 503 cylindrical piles, and about 30,000 cu. yd. of reinforced concrete, there being a total of 3500 tons of reinforcement. The fenders contain 1350 tons of steel. Construction commenced in the Spring of 1950 and is now complete.

The pierhead is to accommodate four ocean tankers of up to 39,000 tons and with a maximum length of about 700 ft., requiring a minimum depth of water of 34 ft. The principal parts are four berthing islands approximately parallel to the coast and spaced at 800 ft. centre to centre; a pierhead trestle 2470 ft. long parallel to, and 130 ft. behind the front of, the berthing islands; four trestles connecting the berthing islands to the pierhead trestle; two dolphins at each end of the pierhead trestle; the pumphouse area at the outer end of the approach, comprising a parking area, a dockmaster's building, a substation and transformer platform, and a pumphouse from which salt water is pumped to the

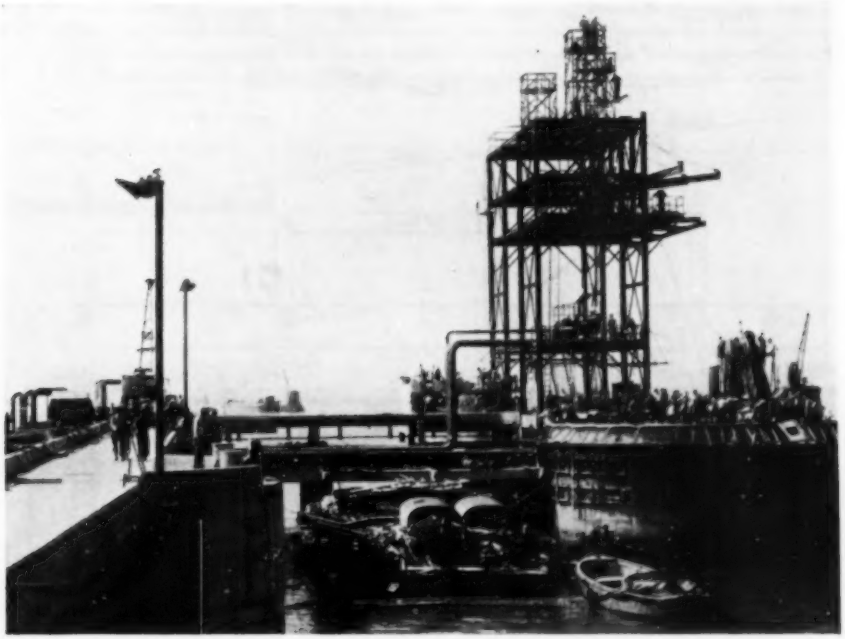


Fig. 1.—Part of Pierhead Trestle, and a Berthing Island nearing Completion.

refinery ; a berth for coasting ships 250 ft. long and three landings for launches on the landward side of the pierhead.

The main parts of the pierhead and the structures on the pumphouse area are supported on hollow cylindrical piles, and the approach is on rectangular piles. The pierhead trestle carries a 12-ft. roadway and two tiers of pipes. Sand-filled cylindrical caissons are provided as strong-points in the pier to resist the large horizontal forces due to the mooring and berthing of vessels. There are twenty-two caissons, that is one at each end of each berthing island and three in each space between the berthing islands, four used as dolphins, and one in the berth for coasting vessels. There are two bollards on each of the caissons between the berthing islands and on the southern dolphin, one on each of the other caissons of the pierhead trestle, two on each of the caissons on the berthing islands, and two on the deck between these caissons. The berthing islands have heavy steel fenders with spring buffers, and a fender of similar type is being provided on the southern dolphin. Sections through the approach, the pierhead trestle and the strong-point of the berthing islands are shown in *Figs.* 3 to 5.

The cylindrical piles have an outside diameter of 32 in., an inside diameter of 20$\frac{1}{2}$ in., and a wall 5$\frac{3}{4}$ in. thick, and are designed to resist vertical loads and moderate horizontal forces such as wind pressure on the trestles. Tests show that the piles can be assumed to be fixed about 10 ft. below the sea-bed.

The cylindrical caissons, the heights of which are from 60 ft. 6 in. to 67 ft., have bottom slabs stiffened by beams. The cylindrical form of the walls is the most economical as it results in the smallest bending moments from the filling. Two sizes of caisson are used, one of 36 ft. outside diameter with 9-in. walls, and one of 40 ft. diameter with 10-in. walls.

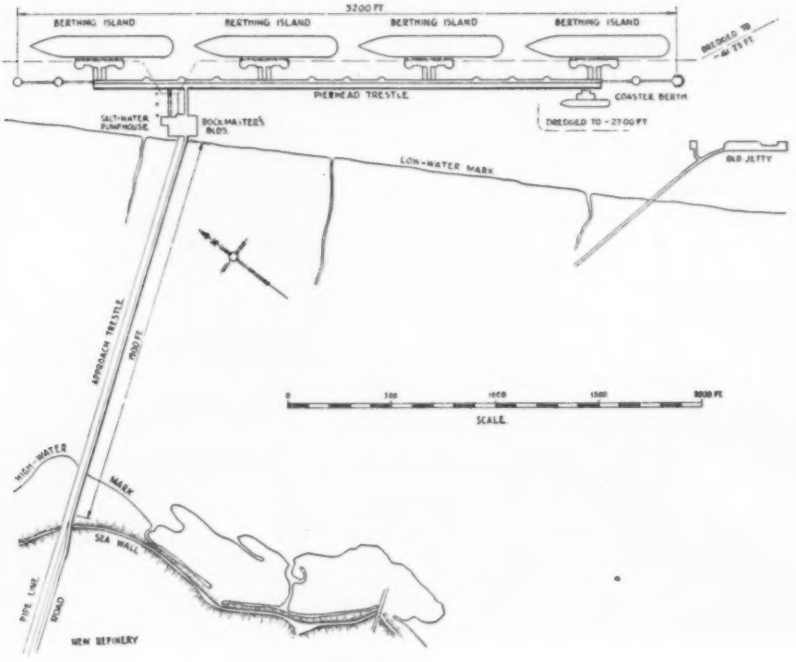


Fig. 2.

Berthing Islands and Connecting Trestles.

Each berthing island has two 40-ft. caissons placed at 196 ft. centres with a trestle on cylinders between them, and another trestle connects the island to the pierhead trestle. A deck-slab generally 12 in. thick covers the caissons and trestles between the caissons at the same level as the road on the connecting trestle. The pipes are carried on beams 8 ft. 5 in. below the deck. The deck on the trestle is 30 ft. wide at the caissons and 35 ft. 8 in. wide at the centres of the islands. For a width of 3 ft. 10½ in. at each side the slab is 15 in. thick. Kerbs 12 in. high are provided around the deck, and 18 in. high along the road on the connecting trestle. An expansion joint is provided between the connecting trestle and the pierhead trestle, but the berthing island and the connecting trestle are a single structure without joints, most of the horizontal forces on them being resisted by the caissons. Calculation showed that thermal expansion and contraction of the deck of the berthing islands will not cause any large additional stresses, as the caissons can deflect due to elastic deformation of the sea-bed.

The frames of the berthing-island trestle are precast and each is supported on two cylindrical piles. The tops of the frames are connected by the deck-slab and the lower parts by two struts and a gangway 2 ft. 6 in. wide. The struts and the gangway were also precast. The frames are at 20 ft. centres in the three central spans and at 16 ft. centres in the three spans at each end. The four frames at the centre are of heavier construction than the others.

Each precast frame in the connecting trestle is supported on four cylindrical piles at 20 ft. centres, the frames also being at 20 ft. centres. The beams support the pipes, and cast-in-situ beams between the two central piles carry the road. Connection is made to the frames of the berthing islands by four precast struts, one above each row of cylindrical piles.

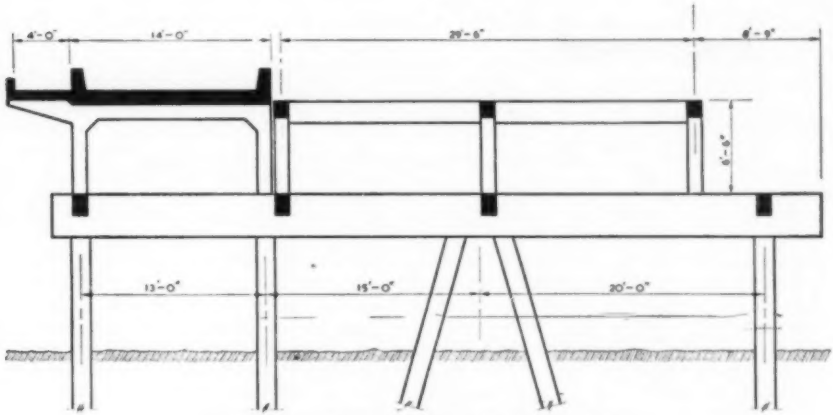


Fig. 3.—Cross Section of Approach.

A lower slab near the top of each caisson (*Fig.* 5) acts as a cover to the sand filling and stiffens the walls against the forces from the fenders as the spring buffers are attached at this level. The wall of the caisson between the lower slab and the top deck was cast in situ and forms a circular space between the two slabs. The space is divided by a wall which acts as a deep beam to support the secondary beams of the two slabs. Access is provided to the space by openings in the wall.

Four fenders are placed on the front of each berthing island, two between and two on the caissons. Each consists of a welded steel frame faced with 12-in. square greenheart rubbing strips supported on 24-in. by 12-in. steel piles. The frames are assembled away from the

site and lowered complete in front of steel piles, to which they are connected by welding above water and by bolts below water. Coil-spring buffers are placed between the piles and the concrete structure. The rubbing strips extend from the underside of the deck to about 15 ft. below low water. Each steel pile weighs about 7 tons and was driven by a floating pile-driver. The frames weigh about 55 tons each and were carried while suspended from a floating crane from the works at Southampton, where they were made, directly to their position in the jetty.

cast struts in addition to the connection formed by the road and the precast gangway. The frames, which are generally at 20 ft. centres, resist the wind forces acting at right-angles to the axis of the trestle, so that bending moments are produced on the cylinders. Forces parallel to the axis of the trestle, such as those due to friction between the pipes and their supporting frames caused by expansion and contraction, are transmitted to the caissons through the struts and the road slab, and the reinforcement in the struts is therefore greatest at the caissons and least at the expansion joints.

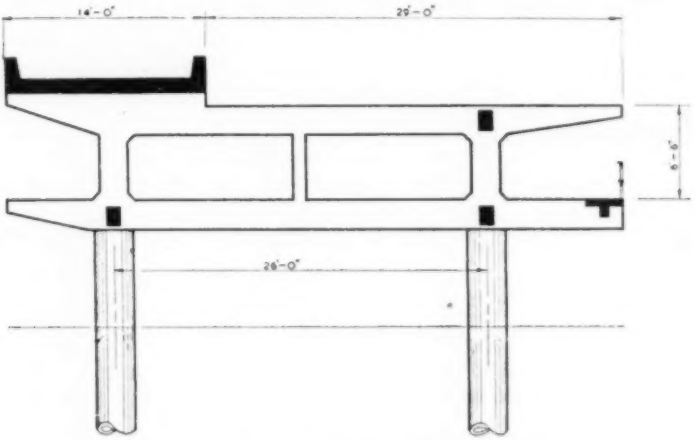


Fig. 4.—Cross Section of Pierhead Trestle.

Pierhead Trestle.

The pierhead trestle is 2470 ft. long and connects the berthing islands with the approach. Transverse beams with overhanging ends carry pipes at two levels as in *Fig.* 4. A road is carried on the top beams on the seaward side and a gangway on the landward side of the lower beams. An expansion joint is provided between the pierhead trestle and the approach, and the pierhead trestle is divided by expansion joints into sections from 230 ft. to 300 ft. in length. There is an expansion joint in each space between the caissons, so that each section is tied to one caisson only. The caissons are 36 ft. diameter.

Each frame of the trestle was precast, is supported on two cylindrical piles, and is connected longitudinally by pre-

The caissons are designed to resist the pulls on the bollards and the forces carried over from the part of the trestle to which the caisson is connected. The pipes are anchored to the concrete structure at some of the caissons, and forces due to the anchorage are thereby added to the forces previously mentioned.

Dolphins.

The two dolphins at the northern end and one dolphin at the southern end of the pierhead trestle are more or less of the same design, being a 36-ft. diameter caisson covered by a concrete deck. A bollard on the seaward side of the caisson is anchored to a block of concrete under the deck. The other dolphin at the southern end is a turning dolphin of special design, being a 40-ft. diameter

caisson with a concrete deck, two bollards on the seaward side, and a large fender comprising five spring-fenders similar to those on the berthing islands.

Berth for Coasting Vessels.

The deck of the berth for coasting vessels on the landward side of the pierhead trestle is supported on a 36-ft. caisson and six cylinders filled with concrete. A fender of 16-in. by 12-in. steel piles with 12-in. by 12-in. steel walings and rubbing strips of greenheart is fixed along the front, which is 86 ft. long.

Substructure of the Pumphouse Area.

The dockmaster's building, which is 94 ft. by 33 ft. and of two stories, is on a concrete deck alongside the road. The parking area in front of the building is 108 ft. by 38 ft. The road at the outer end of the approach is similar to that on the pierhead trestle. The pipes from the approach are carried under the deck of the parking area and on the outer end of the approach to the pierhead trestle. The substructure of the dockmaster's building, the parking area, and the outer end of the approach is separated by expansion joints from the pierhead trestle, the approach, the substation, and the pump-house, and is supported on hollow cylindrical piles. Horizontal forces from wind and friction between the pipes and their supports are resisted by the bending of the piles. Large capping beams are placed at the lower level on the hollow piles, which with the beams form a series of rigid frames. The decks are supported on 16-in. square columns extending above every hollow pile, extra columns between the piles being provided under the beams carrying the walls of the dockmaster's building. The columns are designed to resist the vertical loads combined with the horizontal forces from wind and friction of the pipes. The lower tier of pipes under the parking area is supported directly on the capping beams, and the upper tier is carried on beams supported on the columns.

The deck of the substation and transformer platform is at the same level as the parking area, but the deck of the pumphouse is lower. The capping beams below the deck of the substation and transformer platform are a little lower than the pumphouse deck, to which they

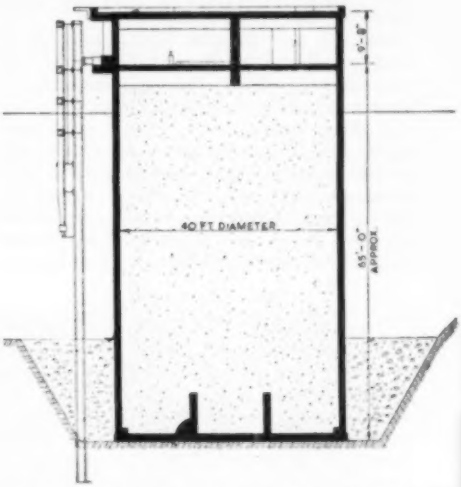


Fig. 5.—Cross Section of Strong-point of Berthing Island.

are connected. The pumphouse is supported directly on hollow piles, and the decks of the substation and transformer platform are carried on columns supported on piles. The basin below the deck of the pumphouse and the forebay is enclosed by steel sheet-pile walls. Water enters the forebay through a concrete flume extending to the pierhead trestle. The sheet-pile walls, the design stresses in which are small to allow for the risk of corrosion, are designed to resist differences between the pressure of water and earth outside and inside the walls, the horizontal reactions to which are provided by the deck and a layer of concrete on the bottom of the basin. The vertical and horizontal loads from the superstructure due to dead and live loads, wind, and the pressure of water currents are assumed to be resisted by the piles. It is not intended to fill the trench around the walls, but possible filling by sedimentation is assumed in the design. The pump basin is 27 ft. by 69 ft. and the forebay 8 ft. by 66 ft. The sheet piles are 76 ft. long and were driven by a floating pile-driver 35 ft. below the dredged level of the sea-bed. The 2-ft. slab at the bottom of these chambers was deposited through an 8-in. tremie.

The concrete flume is 130 ft. long,

10 ft. wide and 6 ft. high. It is supported on four frames each comprising two hollow piles and two precast beams one below and one above the flume. The flume was cast in a dry dock, the ends were closed by timber bulkheads, and it was towed to the site. Two frames placed about one-fifth of its length from the ends carry the weight of the flume, and the other two frames resist horizontal forces only; this arrangement allows the bending moments on the flume due to dead load to be calculated more exactly.

The Approach.

The approach is 1900 ft. long and is divided by expansion joints into sections of 200 ft. The structure comprises 99 frames, generally at 20 ft. centres, and a reinforced concrete substructure. Each frame is supported on 14-in. by 16-in. reinforced concrete piles varying in length from 55 ft. to 85 ft. A heavy piled anchor-frame is provided at the centre of each section. For the first 800 ft. there are four vertical piles in each frame, the anchor-frames having three vertical piles and 18 inclined piles. Beyond 800 ft. there are five piles, of which two are inclined, in an ordinary frame. The piles are connected transversely at the top by a cast-in-situ capping beam. Strutting longitudinally is effected by precast reinforced concrete beams connected at each end to the capping beams. The cast-in-situ superstructure comprises a 14-ft. road, a 4-ft. gangway, and reinforced concrete beams supporting the pipes. The road and beams are carried on columns supported on the capping beams.

Design Data.

Concrete of two qualities was used, one having a strength at 28 days of 3500 lb. per square inch for members above water, and the other having a strength of 4500 lb. per square inch for members below water, such as the cylinders, piles, the walls and bottoms of the caissons, and the flume. The thickness of concrete over the main reinforcement is at least 2½ in. below water and at least 2 in. above water. The working compressive stress in concrete of the greater strength is 1570 lb. per square inch and the shearing stress 105 lb. per square inch; the corresponding stresses for concrete of the lower strength are 1350 lb. per square inch and 90 lb. per square inch.

The deck of the parking area, the berthing islands, and roads are designed for an imposed load of 300 lb per square foot. Alternatively the live load on the parking area is a lorry of 40,000 lb. gross weight plus 30 per cent. for impact, and on the road on the pierhead trestle a lorry of 20,000 lb. gross weight. The gangways are designed for a live load of 100 lb. per square foot. As previously mentioned, the structures are also designed to carry the pipes and resist the frictional forces due to the expansion and contraction of the pipes.

The pressure of the wind is assumed to be 25 lb. per square foot, the exposed area being assumed to extend 6·5 ft. above the roads.

The pull from hawsers is assumed to be 60 tons on each bollard. The fenders on the berthing islands are designed to withstand the forces imposed by the berthing of 26,000-tons tankers.

The working tensile stress in mild steel bars is 18,000 lb. per square inch in the main reinforcement in beams and 16,000 lb. per square inch in reinforcement resisting shearing force. The compressive stress in reinforcement in columns is 14,000 lb. per square inch.

In a later number it is hoped to describe some of the methods of construction.

Standards for Cranes.

A NEW British Standard No. 1757 (1951), "Power-driven Mobile Cranes," deals with slewing and non-slewing mobile, semi-mobile, and portable cranes mounted on wheels, endless-tracks, or lorries. British Standard No. 327, Part 1 (1951), "Power-driven Derrick Cranes," which has been revised, deals with Scotch-derrick, guy-derrick, and tower-derrick cranes. Both standards (price 6*s.* each from the British Standards Institution) specify factors of safety for the design of various parts, and requirements of stability and testing are given.

Multiple-Story Buildings with Precast Walls at London Airport.

THE accompanying illustrations show stages in the construction with precast slabs of buildings of two and three stories. The buildings, which are at London Airport, are a departure building (*Fig.* 1), a three-story office building (*Figs.* 3 to 5), and four two-story office buildings. The departure building is 221 ft. 3 in. long and 32 ft. 9 in. wide, and the three-story office building is 121 ft. 3 in. long and 32 ft. 9 in. wide.

Fig. 1.—The Departure Building.

Fig. 2.—Departure Building During Construction

The precast reinforced concrete slabs, which form the exterior walls and are similar to those used for houses as described in this journal for September, 1947, are 7½ in. thick, 5 ft. wide, and 8 ft. 3 in. high in the three-story building and 11 ft. 3 in. high in the lower story of the departure building. The slabs, which have openings for windows where necessary, were made in a factory and taken to the site on lorries from which they were lifted by a mobile crane, having a 50-ft. swan-neck jib, and placed directly in a vertical position around a temporary jig to which they were secured with wire during the placing of concrete between the vertical edges of the slabs. The jig is erected on the floor of the story being built and comprises timber posts and horizontal beams. The posts are at about 10-ft. centres and are held vertically by diagonal steel tubes. The beam is placed at the level of the window heads on steel angles projecting horizontally from the posts. The front face of the beam coincides with the back face of the wall. A slab 11 ft. 3 in. by 5 ft. without an opening weighs 18 cwt., and a similar slab 8 ft. 3 in. high weighs 13·3 cwt. The minimum age of the slabs at the time of erection was three weeks. The proportions of the concrete are 1 : 1½ : 3, the aggregate being graded from ⅜ in. downwards. The concrete was consolidated by vibration.

Fig. 3.—First Story of Office Building.

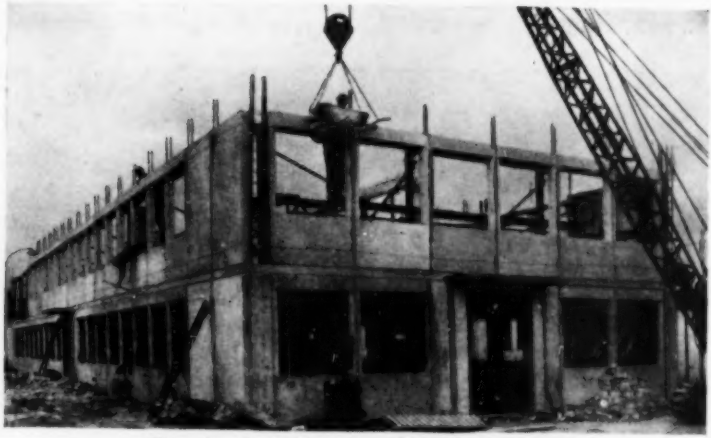


Fig. 4.—Second Story of Office Building.

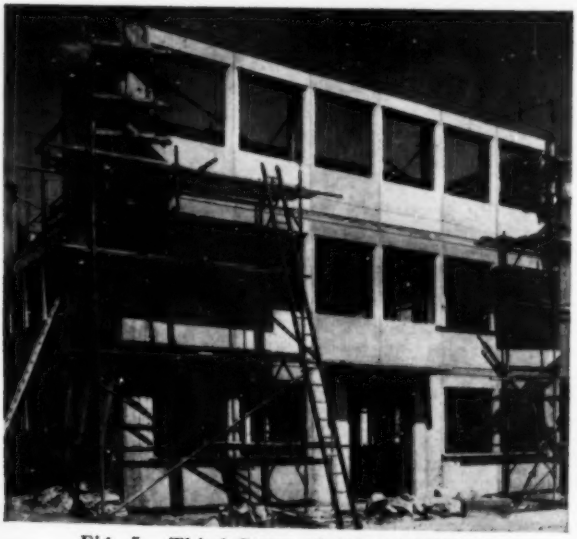


Fig. 5.—Third Story of Office Building.

The vertical edges of the slabs are recessed, two adjoining recesses forming a rectangular cavity in which reinforcement was inserted and which was filled with fine concrete. At each corner of the buildings there is a cast-in-situ column 19 in. by 7½ in. (*Figs.* 4 and 5). Wooden shutters for three sides of the column were wired to the jig, the shuttering for the fourth side being formed by the ends of the slabs. Concrete cast in wooden shutters also forms the surrounds of the doors (*Fig.* 3).

Fixing plugs for steel window frames are embedded in the slabs. Concrete window sills were precast separately. Partition walls are generally of clinker blocks, and metal ties projecting from the joints between two wall slabs secure the partitions to the exterior walls. Similar ties project from the interior columns.

The 6-in. slabs of the upper floors and flat roof were cast on shuttering formed by adjustable steel beams of triangular cross section. The edges of the floor slabs appear externally as a stringcourse, and the roof slabs overhang to form eaves cast in wooden shuttering (*Fig.* 1). The floor and roof slabs are supported by the exterior walls and 9-in. square beams, supported at 10-ft. centres on cast-in-situ columns, approximately along the centre-line of the building. The beams were precast in lengths of 8 ft. 10 in. and have sloping ends which bear on projections having similar sloping faces and cast monolithically with the columns. The faces of the beams were hacked to form a key for plaster. The roof slab is covered with a layer of aerated concrete of an average thickness of 2½ in. and two layers of two-ply bituminous felt.

No scaffolding was used except at the corners. The mobile crane was used to hoist concrete and other materials. The mixture of the concrete cast in situ is 1 cwt. of cement, 2¾ cu. ft. of fine aggregate, and 4¾ cu. ft. of coarse aggregate graded from ¾ in. to ⅛ in. The cast-in-situ reinforced concrete work is designed in accordance with the British Standard Code of Practice No. 114. The reinforcement is mild steel.

The two-story departure building, the area of which is 7246 sq. ft., was completed in sixteen weeks. The first story of the three-story office building, including the first floor but excluding the ground floor and foundations, took three weeks, and the second and the top stories four weeks each, the total time for the construction of this building of 3971 sq. ft. being eighteen weeks, of which seven were occupied in clearing the site and constructing the foundations. The average number of men employed was 160, some of whom were working on the four other office buildings.

The buildings were designed by the Air Ministry in collaboration with the contractors, Messrs. Wates, Ltd., who made the precast members at their own factory.

Periodicals on Prestressed Concrete.

THE progress of the use of prestressed concrete has resulted in the publication of at least two periodicals on the subject. One of these, entitled "Hormigon Elastico," is printed in the Spanish language and has been published in Buenos Aires for nearly three years. No. 1 of a journal entitled "Precontrainte" has now been published in Belgium by Science et Technique, 21 rue Newton, Brussels, under the auspices of Professor G. Magnel. Articles are printed in the language of the author, and abstracts are generally given in English and French. The first article, by Mr. K. P. Billner (U.S.A.), describes a method of anchoring prestressing wires within a concrete member and applying the prestressing force at the middle; this is printed in the English language with a summary in French. An article by M. Willy Kern on the uses of prestressed concrete in the north of France is in French only. M. Henry Lossier gives a long account of the state of compression that can be imparted to concrete by the use of cement with expansive properties; this is printed in French and has a brief summary in English. Tests on the fire-resistance of prestressed concrete are described by Mr. G. Baar (Holland) in the Dutch language, and short summaries are given in English and French. The last two of the 48 pages of this number give a bibliography of recent articles and other publications in America and Germany on prestressed concrete. It is proposed to publish two numbers in the first year, and four thereafter. The annual subscription rate is 140 Belgian francs (about £1 sterling).

Precast Concrete North-light Roof Trusses.

THE roof of part of the factory of Adhesive Tapes, Ltd., at Elstree, comprises precast concrete north-light trusses and purlins supporting asbestos-cement sheeting and glazing. In parts where the fire risk is great the flat roof is a solid cast-in-situ reinforced concrete slab. Permanent exterior walls are of cavity brickwork, but walls which will be demolished when the buildings are extended are of hollow concrete blocks. The walls of the boiler house are mainly of glass between exposed rectangular reinforced concrete frames of concentric on bases generally 4 ft. square. The bottom of the column, which has a rough surface, is set in a cavity in the base in which it is held temporarily by steel gusset plates. The plates are removed after the column has been concreted in the cavity.

To avoid the need to provide struts and ties to resist horizontal thrusts during erection, the frames were considered to be freely supported at this stage of the construction and to carry only their own weight and that of two purlins erected at

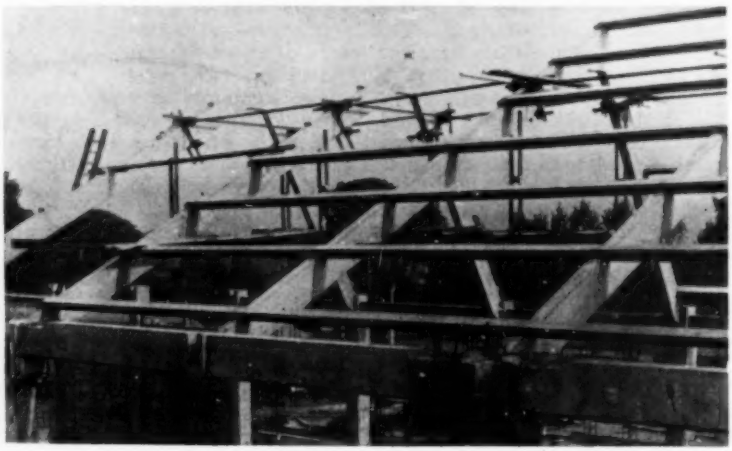


Fig. 1.—Roof under Construction.

41 ft. span and about 25 ft. high supporting a cast-in-situ reinforced concrete roof slab of low pitch.

The area covered by the north-light roof of one building (*Fig.* 1) is about 100 ft. by 133 ft. The reinforced concrete frames (*Fig.* 2) span 33 ft. 4 in., and are at 12 ft. 6 in. centres. Alternate frames are supported on columns arranged on a grid of 25 ft. by 33 ft. 4 in., and intermediate frames are supported on precast concrete valley beams. The frames are 6 in. wide, the more steeply sloping rafter being 13½ in. deep and at an angle of 60 deg. The other rafter is 15 in. deep and at an angle of 30 deg.

The interior columns A (*Figs.* 3, 4, 6), which are 12 in. square and were precast, are designed to act only as props, and are the same time to provide lateral restraint. The ends of the frames rest in pockets in the columns and valley beams, sufficient space being provided to allow the frame to spread so that vertical forces only had to be resisted by the columns. When all the frames had been erected, the joints at the ends of the frames, through which ¾-in. bolts pass, were filled with stiff grout. Subsequent loads due to the remainder of the purlins, roof sheeting, glazing, and other dead and live loads are carried by the frames acting as a series of continuous arches, the horizontal components of the thrusts being transmitted through each frame to the head of the exterior columns.

The exterior columns B and C were cast in situ and act as vertical cantilevers, their bases being designed accordingly.

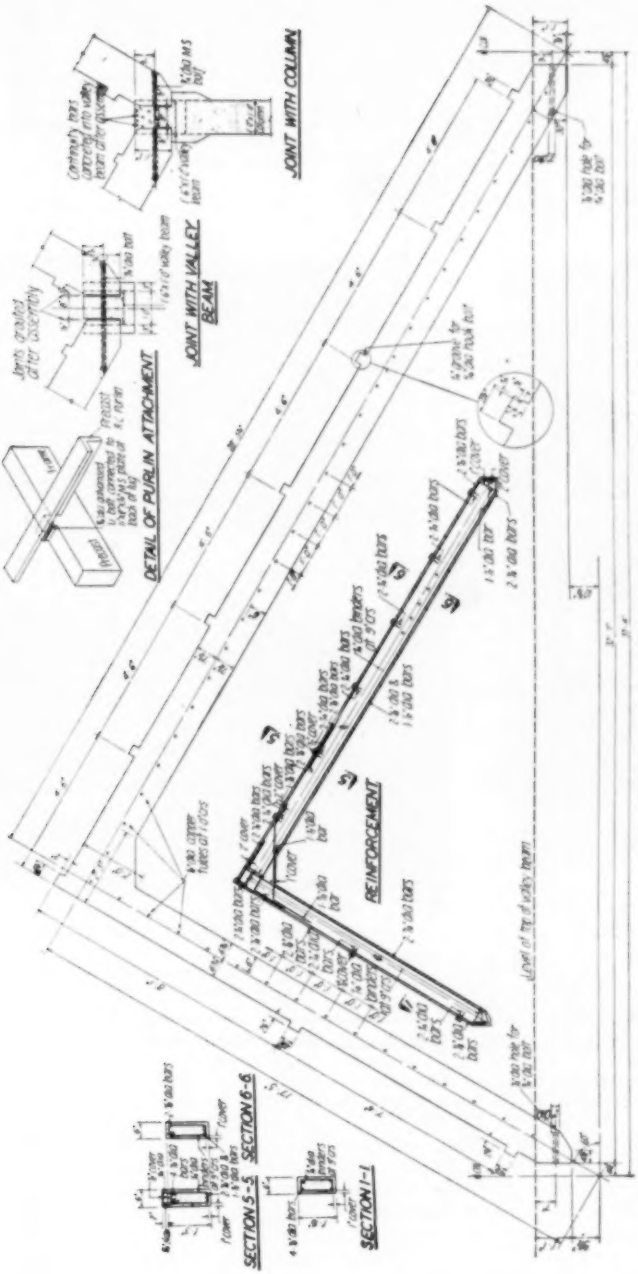


Fig. 2.—Details of Precast Trusses.

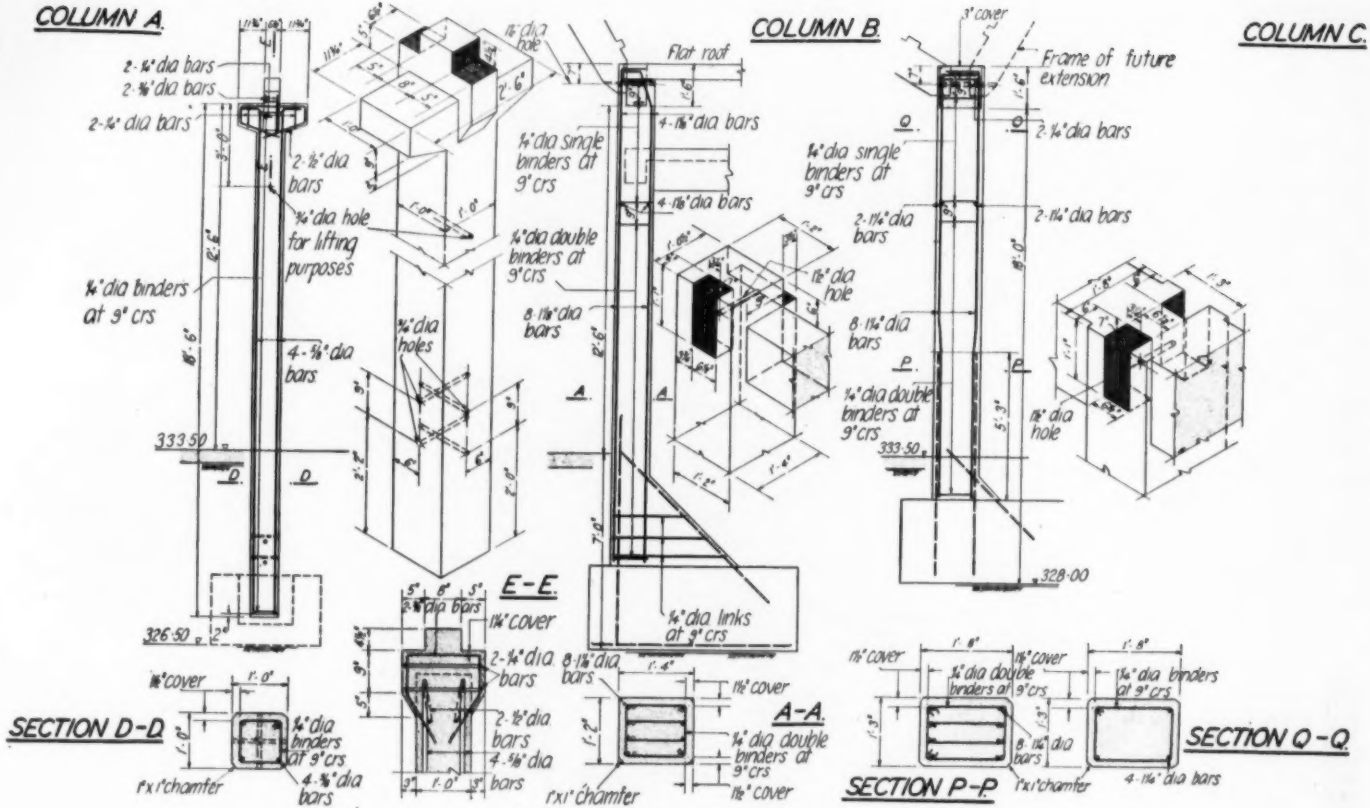


Fig. 3.—Details of Columns.

Typical sizes are: Column B, 16 in. by 14 in., 2 ft. 3 in. eccentric on a base 7 ft. 6 in. by 4 ft.; this column carries part of the flat roof of the office in addition to the north-light roof. Column C, 20 in. by 15 in., 2 ft. 6 in. eccentric on a base 9 ft. by 5 ft.; this column is designed to carry a future extension of the north-light roof (*Fig.* 3).

The precast reinforced concrete purlins are ell-section with 1-in. stiffening gussets at 3-ft. centres. Generally the depth is 6½ in., the width 5 in., and the thickness of the web and flange 1½ in. Details of the connection to the frames are shown in *Fig.* 2. The precast valley beams are 18 in. deep, 12 in. wide, and 24 ft. 5 in. long. They are supported on corbels on the heads of the columns (*Fig.* 1), to which they were clamped temporarily during erection. Two additional ⅞-in. reinforcement bars embedded in concrete cast in situ over the columns ensure continuity.

The volumetric proportions of the concrete are 1 : 2 : 4 in the columns and 1 : 1½ : 3 in the frames, purlins, and valley beams. The minimum crushing strength at 28 days is 3000 lb. per square inch for 1 : 2 : 4 concrete and 3750 lb. per square inch for 1 : 1½ : 3 concrete. The working stresses are in accordance with the British Standard Code No. 114 (1948).

The precast columns, each of which weighs about 1½ tons, were made at the site and were erected by an 8-tons mobile crane. For unloading the columns were slung from two points (*Fig.* 4), and for erection a steel shackle was attached near the head (*Fig.* 6). The part of the column embedded in the foundation was roughened and the space around the bottom filled with 1 : 2 mortar. The shuttering for the columns cast in situ was lined with resin-bonded plywood, and had steel clamps. The moulds for the precast members were lined with hardboard and in some cases with plywood.

The frames were cast on the site in stacks of ten (*Fig.* 5) in a deep timber mould lined with hardboard, separation of

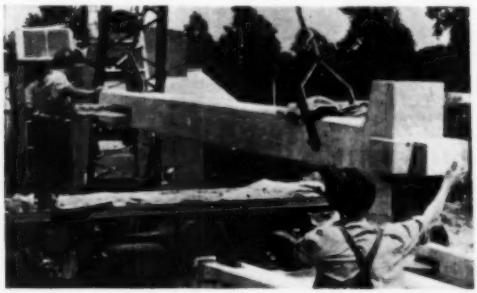


Fig. 4.—Lifting a Column.

Fig. 5.—Lifting a Truss.

Fig. 6.—Erecting a Column.

the castings being effected by a sheet of hardboard. No frame was erected until at least three weeks after casting. Before being lifted off the stack, each frame was stiffened by a tie of double steel channels to prevent the rafters spreading and to relieve the frame of stress during handling. Each frame weighs about 2½ tons, and was transported and erected by the crane. The crane slings were attached at three points, timber packing being provided to prevent damage to the concrete.

The architect is Mr. E. F. Peat, A.R.I.B.A., and the designers of the reinforced concrete work are the British Reinforced Concrete Engineering Co., Ltd. The contractors are Messrs. John Laing & Son, Ltd.

Standard for Testing Aggregates.

BRITISH Standard No. 812 (1951), " Sampling and Testing of Mineral Aggregates, Sands, and Fillers " (price 6s. from the British Standards Institution), has been revised to correlate the methods of testing aggregates for concrete, mortar, asphalt, and bituminous macadam. Instructions for sampling and methods of sieving are given in greater detail. A new method of determining the amount of clay, silt, and dust is given, but the two methods given in the previous issue are retained. A new thickness gauge is made of thin plate so that curved particles of aggregate do not cause so much difference in the results of sieving and gauging tests. The absorption and density tests of specimens of rock are omitted, but the corresponding tests on aggregates are altered and require periods of dry and soaking of only 24 hours each instead of 72 hours. Although the abrasion test is not considered to be satisfactory it is retained pending the development of a better method. A new impact test on ½-in. chippings is specified. Attrition tests on 1-in. and ½-in. stone are omitted, and the crushing-strength test is now applied only to solid rock, but the test to determine the aggregate crushing value is retained. Tests for slag and clinker are omitted as they are given in other standards.

Fire Resistance of Prestressed Concrete.

IN " Precontrainte," No. 1, 1950, Mr. G. Baar describes a test to determine the resistance to high temperature of a prestressed concrete tee-beam, 38 ft. long and 25 in. deep, prestressed by a Blaton-Magnel cable of twenty-four 0·2-in. wires. The beam, the age of which was six months, was placed in a firebrick trough and heated from the bottom by twenty-two gas burners. Thermo-couples were embedded in the beam, which was loaded with concrete blocks equal in weight to the design load.

The highest temperature reached at the bottom of the rib was 1310 deg. F., the temperatures at the level of the cable at the same time being from 450 deg. F. and 520 deg. F., and at the top flange 212 deg. F. Deflections up to 7·6 in. occurred. After the test no deformation or displacement of the wires or anchorages was observed ; the ends of the cables were not damaged, nor was the quality of the wire affected.

The author's conclusions from this test and other investigations are : (1) A cover of concrete 2 in. thick and efficient grouting provide sufficient protection for the cable at temperatures up to 1475 deg. F. (2) The compressive and tensile strengths of high-quality concrete decrease considerably when the concrete is exposed to high temperatures. (3) The behaviour of a specimen during a fire-resistance test is not a dependable guide to the behaviour of a specimen of very different size. (4) The additional deflection during a fire-resistance test depends largely on the magnitude of the load carried during the test. (5) Complete failure of a prestressed concrete member is not expected in case of fire, but large deflections are more likely to occur than in a reinforced concrete member.

Change of Address.

Messrs. Leslie Turner & Partners, consulting engineers, have removed to 6 Grove Road, Sutton, Surrey (telephone Vigilant 0042).

Design of a Helical Staircase.

THE design of a helical reinforced concrete staircase at the Royal Festival Hall, London, was described by Mr. E. O. Measor and Mr. D. H. New in the Journal of the Institution of Civil Engineers for May, 1951, from which the following is abstracted by permission.

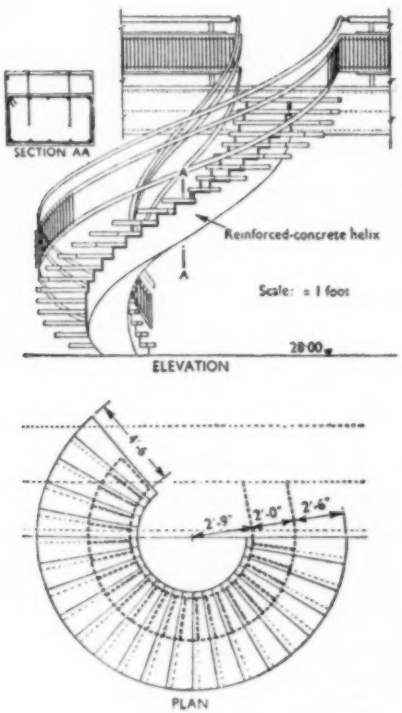


Fig. 1.

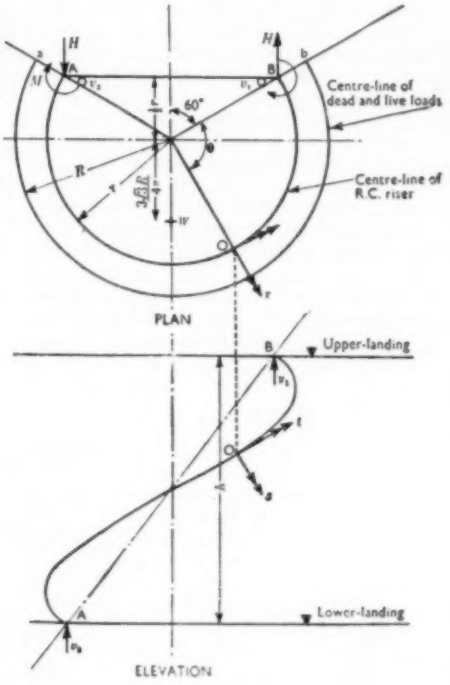


Fig. 2.—Loading Diagram (simply supported at ends).

The staircase (*Fig.* 1) is 12 ft. high and has an internal radius of 2 ft. 9 in. The helical rib is rectangular in cross section and is 2 ft. wide and 12½ in. thick. The treads are about 5 ft. long and are precast in terrazzo. They are bolted to the rib beyond which they cantilever about 2 ft. 6 in. This design gives an appearance of lightness and provides a cross section suitable for resisting torsion. The stresses in the rib due to the uniform load are calculated by the strain-energy method. The effects of uniform and concentrated loads were checked by an elastic model. The calculations are as follows.

It is assumed that the total load on each tread (including the weight of the appropriate part of the helical rib) is concentrated at its centre of gravity at radius R from the axis of the helix. The problem for distributed load, therefore, is that of a helix of radius r subjected to a line load applied on a concentric helix of radius R. The helix turns through an angle of 240 deg. The solution depends on the restraint at the ends of the helix. It is seen in *Fig.* 2 that if the ends are simply supported, that is, no bending moments or torque are applied, the total load W produces an unresisted moment about the axis AB. Therefore the

simplest system of support comprises equal and opposite horizontal forces H and vertical forces V_1 and V_2 at points A and B respectively, and a moment about axis AB. The division of the vertical load between V_1 and V_2 depends on the relative stiffness of the upper and lower supports; it is assumed that $V_1 = V_2 = V$.

The bottom of the staircase is supported by a slab which provides full restraint against rotation about a vertical axis, but which offers little restraint against rotation about horizontal axes. The top is supported by a shallow beam which resists fully rotation about a vertical axis and probably offers some resistance to torsion. For simplicity of calculation, it is assumed first that the only forces acting at the ends of the helix are the vertical reactions V, the horizontal reactions H, and moments M about vertical axes.

If the total load per unit length on plan applied at radius R is w, the total load W is $\frac{4\pi}{3}Rw$, which acts at the centroid of the arc ab, that is, at a radius of $\frac{3\sqrt{3}}{4\pi}R$ on the axis of symmetry. Resolving the forces vertically,

$$V = \frac{W}{2} = \frac{2\pi Rw}{3}.$$

Taking moments about the horizontal axis through A,

$$Hh = W\left[\frac{r}{2} + \frac{3\sqrt{3}}{4\pi}R\right].$$

Therefore, $$H = \frac{W}{h}\left[\frac{r}{2} + \frac{3\sqrt{3}}{4\pi}R\right].$$

Taking moments about the axis of the helix,

$$2M = 2Hr \sin 60 \text{ deg}.$$

Therefore, $$M = \frac{\sqrt{3}}{2}\frac{Wr}{h}\left[\frac{r}{2} + \frac{3\sqrt{3}}{4\pi}R\right].$$

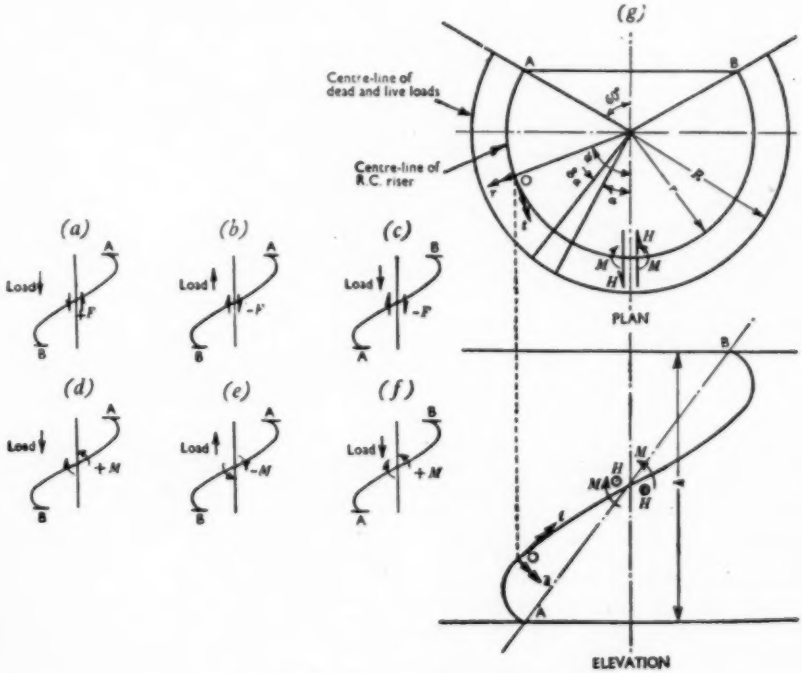


Fig. 3.—Loading Diagrams.

Therefore, with a system of orthogonal axes at any point O on the helix, the bending moment about O_r is

$$\frac{3}{4\pi}Hh\theta\cos\left(\theta-\frac{\pi}{6}\right)+2wR^2\sin^2\frac{\theta}{2}-\frac{2\pi}{3}Rrw\sin\theta \quad . \quad (1)$$

The bending moment about O_s is

$$-Hr\cos\left(\theta-\frac{\pi}{6}\right)\cos\phi+\sin\phi\left[\frac{3}{4\pi}Hh\theta\sin\left(\theta-\frac{\pi}{6}\right)+wR(r\theta-R\sin\theta)-\frac{2\pi}{3}Rw(r-r\cos\theta)\right] \quad . \quad . \quad . \quad . \quad (2)$$

The bending moment about O_t is

$$\cos\phi\left[\frac{3}{4\pi}Hh\theta\sin\left(\theta-\frac{\pi}{6}\right)+wR(r\theta-R\sin\theta)-\frac{2}{3}\pi Rw(r-r\cos\theta)\right]+Hr\cos\left(\theta-\frac{\pi}{6}\right)\sin\phi \quad . \quad . \quad . \quad . \quad (3)$$

in which ϕ is $\tan^{-1}\frac{3h}{4\pi r}$.

The numerical values are : $R = 4{\cdot}67$ ft., $r = 3{\cdot}75$ ft., $h = 12$ ft., and $w = 1161$ lb. per foot. Substitution in equations (1), (2), and (3) gives the bending moments about the principal axes and the torsion at all points in the helix.

For non-uniform load the method applies if w is given different values according to the loads on the various parts of the helix. An investigation made of the case when only half of the staircase is loaded showed that at no point do the stresses exceed those due to the total uniformly-distributed load.

The calculation assumes the least restraint at the ends of the helix, but since the supports exercise considerable, but indeterminate, restraint, the staircase is also analysed on the assumption that the ends are completely fixed. When the staircase is uniformly loaded, the analysis is much simplified from consideration of symmetry. For example, a vertical shearing force F can exist at the central section of the helix when the load acts downwards [*Fig.* 3 (*a*)]. If the direction of the load is reversed, as in *Fig.* 3 (*b*), the direction of this shearing force is reversed so that the vertical shearing force at the central section is $-F$. If the helix is then rotated through 180 deg. about the radial axis of symmetry through the midpoint, the case in *Fig.* 3 (*c*) is obtained, when the load acts downwards and the vertical shearing force is $-F$. But case (*c*) is identical with case (*a*) because of the symmetry of the helix, so that, while in case (*a*) a downward load produces a shearing force $+F$, it follows that the same load produces a shearing force $-F$ in case (*c*). Therefore $+F = -F$, that is, $F = 0$, and in consequence such a shearing force cannot exist at the midpoint.

Considering the case of a possible bending moment M about the radial axis through the midpoint of the helix [*Fig.* 3 (*d*)], if the load is reversed case (*e*) is obtained and, if the helix is rotated about the radial axis of symmetry, case (*f*) follows. Comparing cases (*d*) and (*f*), $+M = +M$, and it is therefore possible for M to exist.

By considering similarly all other possible stress resultants at the midpoint, it can be shown that the only possible forces at this point are the bending moment about the radial axis M and the radial shearing force H, which can be treated as redundants [*Fig.* 3 (*g*)], and the problem solved by the strain-energy method.

At any point O on the centre-line of the helix, the bending moment about the horizontal radial axis O_r is given by

$$M_r = -\left[M\cos\psi+r\tan\phi H\psi\sin\psi+\int_0^{\psi}wR^2\sin(\psi-\alpha)d\alpha\right]$$
$$= -\left[wR^2+(M-wR^2)\cos\psi+r\tan\phi H\psi\sin\psi\right] \quad . \quad . \quad (4)$$

Similarly, the bending moments about the axes O_s and O_t perpendicular to O_r are given by

$$M_s = \left[-wRr\psi+(wR^2-M)\sin\psi+r\tan\phi H\psi\cos\psi\right]\sin\phi+Hr\sin\psi\cos\phi \quad . \quad . \quad . \quad (5)$$

$$M_t = \left[-wRr\psi+(wR^2-M)\sin\psi+r\tan\phi H\psi\cos\psi\right]\cos\phi-Hr\sin\psi\sin\phi \quad . \quad . \quad . \quad (6)$$

The strain energy in half of the helix is then given by

$$U = \int_0^{2\pi} \left[\frac{(M_r)^2}{2EI_1} + \frac{(M_s)^2}{2EI_2} + \frac{(M_t)^2}{2CJ} \right] r \sec \phi \, d\psi \quad . \quad (7)$$

where I_1 and I_2 denote the principal moments of inertia of the cross section of the helix, CJ the torsional rigidity, and E the modulus of elasticity.

Substituting the results obtained from equations (4) to (7) in

$$\frac{\partial U}{\partial M} = O \text{ and } \frac{\partial U}{\partial H} = O,$$

two linear simultaneous equations are obtained for H and M. These values are substituted in equations (4) to (6) to obtain the principal bending moments and the torsion at every point.

Since this method depends on similarity, it is applicable only to symmetrical load. The analysis of asymmetrical load is more laborious owing to the greater number of redundants which do not vanish at the midpoint of the helix. The model was used to check that asymmetrical load did not cause greater moments at any point than symmetrical load.

As the end conditions are probably nearer to simple support than to fixity, the staircase was designed to resist the moments obtained for simple support, extra reinforcement being provided where the analysis for complete fixity indicated that fixity might produce additional stresses.

[The authors inform us that they are indebted to Mr. V. A. Morgan, A.M.Inst.C.E., for pointing out some discrepancies in the equations as originally published, which have been corrected in this abstract.—ED.]

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New Cement Works at Shoreham.

A NEW cement works has been built adjacent to the old cement works at Beeding, near Shoreham, Sussex, of the British Portland Cement Manufacturers, Ltd. The new works comprises two 350-ft. kilns and ancillary plant and structures constructed in a chalk quarry, the committee of the Institution of Civil Engineers for such structures. The new works, which has a capacity of 350,000 tons of cement a year, has been in operation since January last.

The quarry in which the new works is built was formed by excavation for

Fig. 1.—New Cement Works at Shoreham.

and offices, canteen, packing plant, railway sidings together with a wagon-tippler and weighbridge on the site of the old works adjoining. The old and the new works are separated by a road as shown in *Fig.* 1. All the structures, some of which are shown in the accompanying illustrations, are designed in accordance with the British Standard Code of Practice for Reinforced Concrete No. 114, or, in the case of liquid-containing structures, the recommendations of material used in the old works, and is roughly elliptical, 1000 ft. by 800 ft. The sides rise steeply all round, and at the highest part are about 180 ft. higher than the floor. The foundations of the structures in the quarry are on good chalk, but those on the site of the old works are on made-up ground or on piles bearing on hard chalk. Borings showed that whereas near the road there was good chalk about 15 ft. below ground, nearer the river it was at a depth of 80 ft. or more. The

slope is not uniform, and in one case borings about 30 ft. apart showed a difference of about 15 ft. in the level of the chalk. The office block, packing plant and silos are on piles varying from 14 ft. to 74 ft. in length.

Kiln House and Clinker Store.

The kilns, which are about 10 ft. diameter and 350 ft. long and are inclined at 1 in 24, rotate at about one revolution in 50 seconds, and are supported on reinforced concrete piers (*Fig. 2*), the tallest of which are 34 ft. high. The weight of the kiln and bed-plate produces a load of 200 tons on each pier, and the kilns may also impose on each pier a horizontal longitudinal force of about 50 tons and a horizontal transverse force of about 100 tons at the level of the bedplate due to distortion of the kiln as a result of abnormal conditions such as follow a power failure. The kilns are in a single-story building about 500 ft. long, 87 ft. wide, and 65 ft. high to the eaves. It is of steel construction with a corrugated steel roof with strips of glazing, and with asbestos-cement sheeting and continuous strips of glazing on the sides.

Beside the kiln house is a store building of the same length and 75 ft. wide and separated from the kiln house by an annexe 10 ft. wide containing numerous conveyors. The two buildings are connected by a flat roof over the annexe at eaves level. The wind force on both buildings is resisted by large trestle stanchions on one side of the kiln house. The foundations of the stanchions are only 7 ft. 6 in. long, 5 ft. wide, and 10 ft. deep and are embedded in the chalk, and thus act as cantilevers to which the stanchions are bolted. Advantage is taken of the hardness of the chalk to provide restraint, thus avoiding a large spread footing which would be otherwise required to resist the large bending moments caused by the wind.

The electrostatic dust precipitators, which are inside the kiln house, are in a reinforced concrete structure, of complicated shape, through which the flue gases pass. The temperature of the gases may exceed 500 deg. F., and the concrete is protected by brick linings on vertical faces and hollow tiles on horizontal faces; also, there is an air space between the brickwork and the concrete on the outer walls. There is one precipitator for each kiln, and each is supported on four pairs of columns, each pair being at about 16 ft. centres longitudinally and 25 ft. tranversely. The four central columns are 18 in. by 9 in. in cross section, and the four corner columns, which are most affected by the expansion due to temperature of the casing, are small double columns each 9 in. square. The double columns reduce the stresses due to longitudinal thermal expansion to one-half of what they would be if the columns were 18 in. wide in that direction. A steel platform is carried partly on the precipitators and partly on the kiln-house gable-end steel stanchions, and to permit free expansion of the precipitators due to temperature, and also for movement of the kiln house due to wind, the supports on the precipitators are provided with steel roller-bearings in boxes filled with grease. The dust removed from the gases falls into hoppers at the bottom of the precipitators, and is removed by conveyors.

At the end of the kiln house nearer the road, a reinforced concrete structure about 100 ft. high houses some elevated water tanks and an elevated reinforced concrete coal bunker of just over 700 tons capacity; the spaces between the columns are filled with concrete blocks. The capacity of the bunker provides sufficient storage to keep the kilns operating for about two days, thus avoiding the need to work the coal-plant on Sundays.

The lower part of the store building consists of reinforced concrete retaining

Fig. 2.—Kiln Piers.

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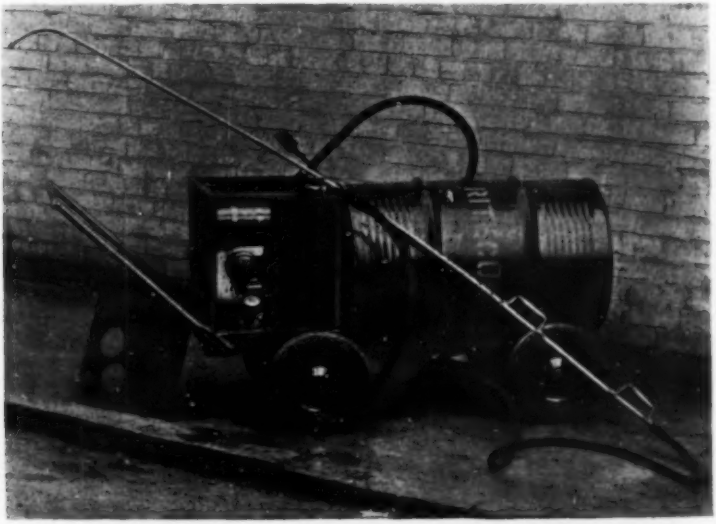

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walls for storage of coal, clinker, and gypsum. There is storage space for about 5500 tons of coal, 1500 tons of gypsum, and 15,000 tons of clinker. The reinforced concrete retaining walls in the clinker store (*Fig.* 3) are up to 40 ft. in height and comprise cantilevered buttresses with walls spanning horizontally between them, a method of construction that proved to be very economical and permitted utilisation of the greatest storage space. Two grab cranes, each of 5 tons capacity, are installed on girders supported on the main steel stanchions. The retaining walls are not connected to the steelwork, since their deflection would cause the stanchions to bend outwards and throw the crane track out of alignment. The reinforced concrete cantilever buttresses are built around the stanchions, leaving a space between the stanchions and the reinforced concrete to allow for movement. The cranes are used for trimming the filling and for taking the materials to bunkers from which are they delivered to conveyors and taken to various parts of the works. The coal-retaining walls are only 16 ft. high because spontaneous combustion might occur if the coal used in cement kilns is stacked to a greater height.

At one end of the store building there are four large elevated reinforced concrete clinker bunkers each of about 260 tons capacity and four gypsum bunkers each of about 100 tons capacity. These bunkers are erected in four pairs, and the conveyors underneath are supplied with clinker and gypsum in the correct proportions. The conveyors deliver to the four grinding mills which, with their motors, are in a building about 140 ft. square adjoining and at right-angles to the main store building.

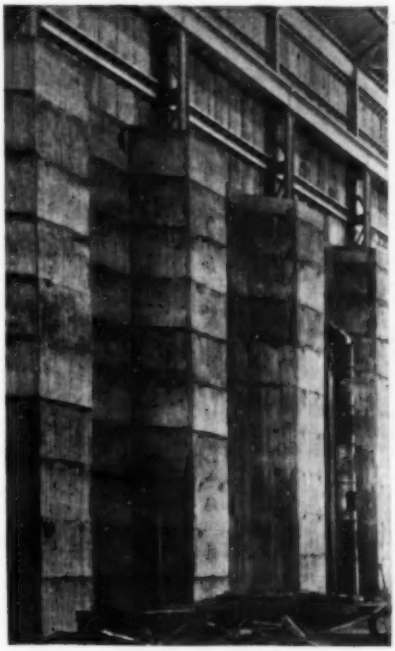

Fig. 3.—Retaining Wall of Clinker Store.

The Chimney.

The chimney (*Fig.* 4) is 300 ft. high and has a reinforced concrete outer wall, the inside and the outside faces of which taper, but the thickness of which varies. At the base the wall is 23 ft. 1 in. diameter outside and 9 in. thick, and at the top it is about 13 ft. diameter and 5 in. thick. A lining of 4½-in. brickwork is provided, with a cavity of about 4 in. betweeen the wall and lining to reduce the temperature of the reinforced concrete wall. The lining is in sections, each 36 ft. 9 in. high, and each supported on reinforced concrete corbels on the inside face of the wall. Holes are provided vertically through the corbels so that the cavities are connected, and inlets through the wall allow free circulation of fresh air which helps to keep the cavity cool. The gases from cement kilns have a high moisture content and are exceptionally harmful to concrete and mortar; the top 15 ft. of the chimney are therefore in brickwork as this is more easily repaired or renewed than concrete.

Cement Silos.

The cement storage silos are of reinforced concrete and consist of three groups each of four circular bins. The bins are 60 ft. high and 26 ft. 6 in. diameter internally. The bottoms, which are flat and have four outlets, are about 10 ft. above the level of the ground floor to provide a space underneath for the conveyors which take the cement to the packing-plant silos. The floor over the bins carries the conveyors which fill the

bins. There are reinforced concrete walls and roof above the floor The height from the ground floor to the roof is 80 feet. The spaces between the four circular bins can be used for a staircase to give access to the conveyor floor. Each bin has a capacity of 1250 tons, so that the total storage capacity of the twelve bins is 15,000 tons. As it was not necessary to provide for storage in smaller quantities, a capacity of 1250 tons per bin was chosen because a circular bin is more economical than a square or rectangular bin, especially in the case of large bins. The height of the silos is such that continuously-moving forms would not be cheaper and climbing shuttering was used. If the silos were 20 ft. higher moving forms would probably have been cheaper.

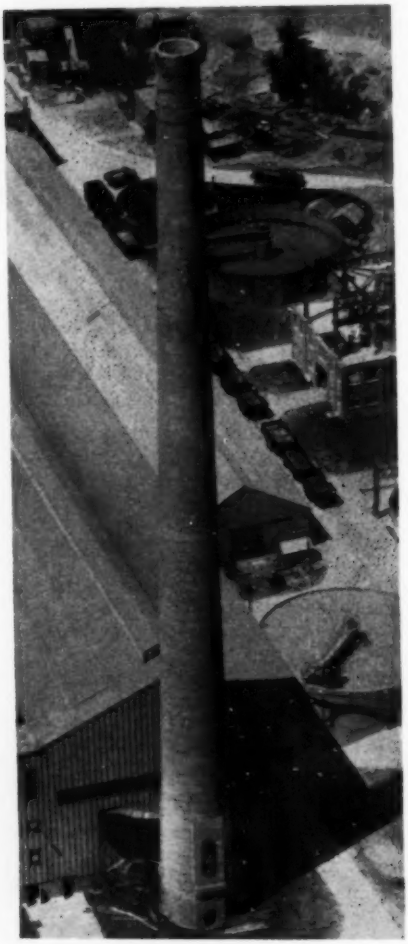

Fig. 4.—Chimney 300 ft. High.

The Slurry Plant.

The chalk is dug from the top of the quarry, whence it is taken to a primary crusher on reinforced concrete foundations inside a crusher house, from whence the crushed material falls down a chute to a pit in the quarry floor. The crusher house is built into the solid chalk; after excavation was commenced some serious faults were found in the chalk, and to avoid the weight of the crusher house and its contents causing a failure along the faults the complete assembly was moved and some of the foundations increased in depth and size. The lines of the faults were traced and the stability investigated by a Swedish method of analysing the stability of earth masses.

The crushed chalk is lifted by a jib crane to an elevated hopper. The crane is supported on a reinforced concrete platform 20 ft. high and 17 ft. square, comprising four 16-in. square reinforced concrete columns, with diagonal reinforced concrete braces, which carry the large beams supporting the platform. The chalk is then conveyed to the washmills, which occupy a site about 160 ft. long and 120 ft. wide. There are three sets of washmills at different levels, supported on reinforced concrete walls and piers. The two primary washmills are at the higher level and are cylindrical in shape and of 35 ft. diameter. The floors of the primary washmills are protected against abrasion by granite setts and walls formed of short lengths of old bull-head rails protect the sides below the screens. The clay comes from a site about three miles away, where it is converted to a slurry and pumped to the works through two 8-in. pipes. It is stored in a tank 66 ft. diameter and 14 ft. 8 in. deep, from which it is fed into the primary washmills and mixed with the chalk. The coarse slurry flows from the primary washmills by gravity to a secondary washmill of about the same size as the primary washmills, where it is further reduced in size and screened to a finer slurry. This slurry flows by gravity to three screening mills,

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(*Continued on page* lviii.)

MISCELLANEOUS ADVERTISEMENTS

(*Continued from page* lvii.)

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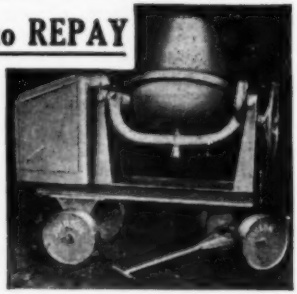

Completely enclosed all-steel machine cut internal gear and pinion (inspection cover). Non-rusting hand wheel. Fully accessible engine housing. Power unit, 1½ h.p. Petter air-cooled, or Lister water-cooled. Diesel radiator cooled Electric motor drive if required. All gears totally enclosed Weight, 9½ cwt. Width, 2 ft. 8 in. Length, 6 ft. Height 5 ft. 2 in. to 5 ft. 6 in. Discharge, 2 ft. 6 in.

Easy loading and discharge. Mixing Drum (heavy cast iron base, heavy gauge cone) pivots through 360 deg.

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Illustrated is PETROL FLEX DRIVE [one of two types] Johnson Internal Vibrator. Vibration is confined to the "needle," thus ensuring long life of motor; imposes no strain on operator. Also Electric Flex Drive and Electric Stiff Shaft models available with 2½ in. and 1½ in. diameter needles. Write for Folder V24.

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also 35 ft. in diameter, where it is further reduced and then pumped to the preliminary mixers. The mixers are two pairs of cylindrical reinforced concrete tanks, the bottoms of which are conical (*Fig.* 5), and are supported on cylindrical walls of the same diameter as the tanks, namely, 23 ft. 7 in. The overall height of the tanks is 54 ft. 6 in. The slurry is mixed in the tanks by air agitation, and is then taken to two final mixers, which are similar in construction to the preliminary mixers, but 27 ft. 1 in. in diameter and 76 ft. high. The slurry is now ready for use, and can be pumped directly to the kilns or to three storage tanks, each 66 ft. diameter and 14 ft. 8 in. deep. The walls of the preliminary mixers are 6 in. thick at the top increasing uniformly to 13 in. at the junction with the conical

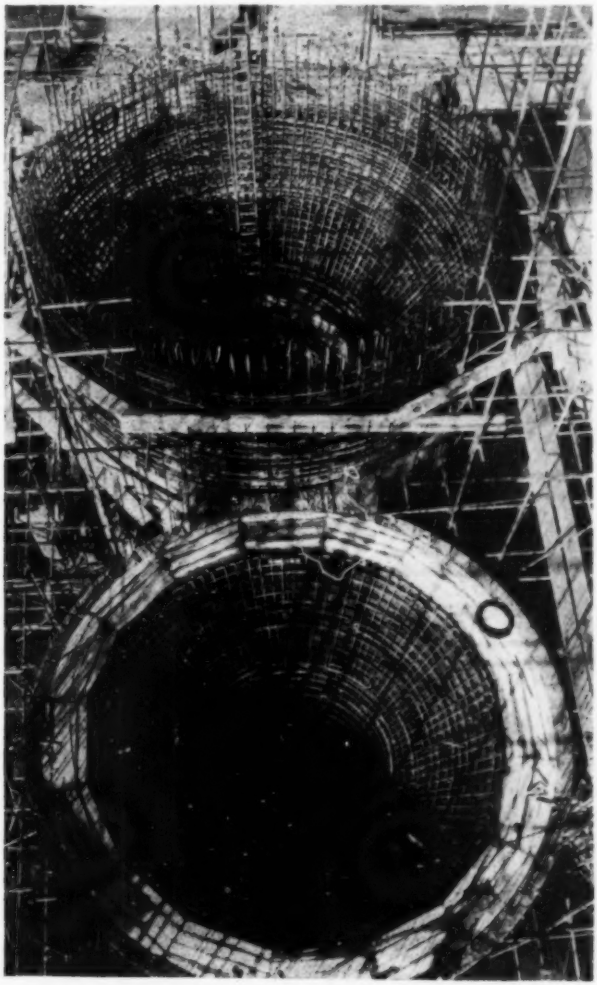


Fig. 5.—Reinforcement in Conical Bottoms of Tanks.

bottoms, the thickness of which decreases to 10 in. at the outlet in the centre of the cones. The corresponding thicknesses of the walls and bottoms of the final mixers are 6 in., 16 in., and 12 in., the concrete being thickest where the hoop tension is greatest. There is in addition to the foregoing a reinforced concrete tank similar to the storage tanks but 70 ft. in diameter in which the wash-water from the flint-washing plant is thickened for delivery to the primary washmills.

The construction joints in the tanks are provided with galvanised steel strips 4-in. wide to prevent the passage of moisture, and the joints were painted on the inside with bituminous paint. The whole of the inside of the tanks was later given a coat of bituminous paint. The slurries which the tanks are designed to contain weigh 76 lb. per cubic foot in the case of the clay slurry and 100 lb. per cubic foot in the case of clay-chalk slurry.

Ancillary Buildings.

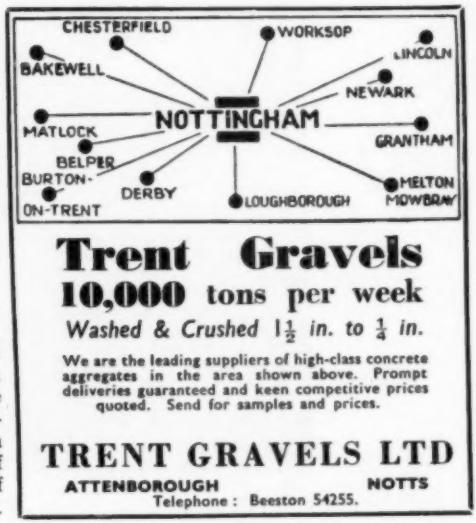
The workshop is a single-story building with a corrugated-steel roof supported on steel trusses and stanchions and with concrete-block walls. It is about 400 ft. long and 50 ft. wide and is heated by floor-type unit heaters connected to a boiler in a small house near one end of the building. There is also a reinforced concrete electricity substation with a suspended floor above for stores and offices. The walls of many parts of these buildings are 18-in. by 9-in. concrete blocks, built in two leaves with an air space between them. The external blocks have a moderately rough texture on the face and are of a light brown hue.

In the sidings the track is generally supported on precast reinforced concrete sleepers, and where the track forms part of a paved area on poor ground it is on a reinforced concrete raft. Where the ground is suitable the track on paved areas is supported on longitudinal reinforced concrete sleepers.

The packing-plant building is 50 ft. by 45 ft. by 73 ft. high, with five stories. The construction is of reinforced concrete columns, beams, floors, and roof, with walls of asbestos-cement sheeting. The loading sheds are of single-story construction with sheeted roofs supported on steel stanchions and occupy an area of about 120 ft. by 120 ft. in one corner of which is the main packing-plant building. The columns of the loading sheds are carried on spread footings.

The office building is of reinforced concrete with concrete-block walls, and consists of a rectangular part 93 ft. by 35 ft. with a curved wing about 50 ft. by 33 ft. jutting out from the side. The ground floor contains laboratories, first-aid room, washing facilities, locker rooms, lavatories, and so on. The first floor comprises the general office and drawing office, and the canteens and kitchen occupy the second floor. The external surface of the walls is covered with cream rendering with the surface lightly rubbed.

The Engineering Department of the Associated Portland Cement Manufacturers, Ltd., was responsible for the planning and general design of the works. The main contractors were Messrs. John Laing & Son, Ltd. Messrs. Bierrum & Partners, Ltd., carried out most of the reinforced concrete work, both as sub-contractors and as direct contractors before the main contract was placed. The structural steelwork was by Messrs. Redpath, Brown, Ltd., and Messrs. J. L. Kier & Co., Ltd., were the contractors for the clay plant. The consulting engineers to the Company, Messrs. Oscar Faber & Partners, were responsible for the design and supervision of the civil engineering works, the whole of which cost approximately £1,000,000.



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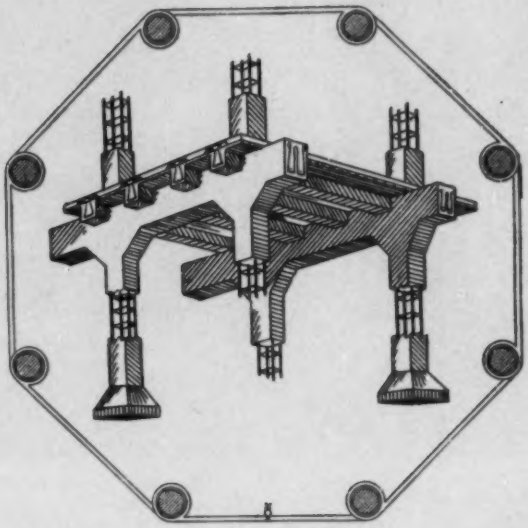

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